Stormwater Site Plan

Issaquah Evergreen Ford
Issaquah, WA

Prepared For:
Evergreen Ford
1500 18th Ave.
Issaquah, WA 98027

Prepared By:
SCJ Alliance
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March 2019

SCJ ALLIANCE
CONSULTING SERVICES

Stormwater Site Plan

Project Information

Project:	**Issaquah Evergreen Ford**
Prepared for:	**Evergreen Ford** 1500 18th Ave. Issaquah, WA 98027 Contact Name: Daniel Rowe Contact Phone: 425-392-6900

Reviewing Agency

Jurisdiction:	City of Issaquah

Project Representative

Prepared by:	**SCJ Alliance** 8730 Tallon Lane NE, Suite 200 Lacey, WA 98516 360.352.1465 scjalliance.com
Contact:	Tyrell Bradley, PE
Project Reference:	SCJ #1883.01 Path: N:\Projects\1883 Strotkamp Architects\1883.01 Evergreen Issaquah Ford\Phase 02 - Construction Documents\Design\Storm\Stormwater Site Plan\2019-xxxx Stormwater Site Plan.docx

PROJECT ENGINEER'S CERTIFICATION

I hereby certify that this Stormwater Site Plan for the Issaquah Evergreen Ford Dealership project has been prepared by me or under my supervision and meets the minimum standards of the City of Issaquah and normal standards of engineering practice. I hereby acknowledge and agree that the jurisdiction does not and will not assume liability for the sufficiency, suitability, or performance of drainage facilities designed by me.

03/05/2019

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1. PROJECT OVERVIEW

The following report was prepared for the Issaquah Evergreen Ford Dealership project in Issaquah, WA. This report was prepared to comply with the minimum technical standards and requirements that are set forth in the *2014 Department of Ecology Stormwater Management Manual for Western Washington (SWMMWW)* and the 2017 Stormwater Design Manual Addendum*.*

Project Proponent:	Evergreen Ford
Parcel Numbers:	2724069084, 2724069086
Total Parcel Area:	3.92 Acres
Current Zoning:	IC – Intensive Commercial
Required Permits:	Grading, Utility, Paving, Building, etc.
Site Address:	6721 30th Ave. SE
Section, Township, Range:	Section 27, Township 24 N, Range 6 W

The proposed Evergreen Ford site is located on two parcels that contain a total of 3.92 acres. The project is located on the south east corner of E Lake Sammamish Parkway SE and 229th Ave SE in Issaquah, WA. The proposed construction includes the 4-story ford dealership building/parking garage, as well as associated parking lot, utilities, frontage improvements, and stormwater improvements disturbing approximately 3.58 acres. Specifically, the proposed site improvements/construction activities for this project include the following:

- Site preparation, grading, and erosion control activities
- Construction of Ford dealership and parking garage
- Construction of parking lot
- Construction of off-site improvements
- Construction/installation of on-site water quality and flow control facilities
- Extension of available utilities (i.e., water, sewer, etc.)

A site vicinity map of the proposed project location is enclosed herein as **Appendix 1**. A worksheet for determining the number of Minimum Requirements for this project per the SWMMWW has been prepared and enclosed herein as **Appendix 2**. Per Table 1-1 from the City of Issaquah 2017 Stormwater Design Manual Addendum, the proposed project is a new development not located within the Central Issaquah Alternative Flow Control area and will created over 5,000 S.F. of new hard surfaces, therefore the project will trigger Minimum Requirements #1-9. Additionally, the pre-developed conditions must be modeled in forested.

Table 1-1 PROJECT SCREENING FOR STORMWATER REVIEW							
	Screening Thresholds[a]			Minimum Requirements[a]			
Project Type[b]	Hard Surfaces		Land Clearing	MR #1-5	MR #6-9	Stormwater Facility Target Surfaces[d]	Pre-Dev Cond.
1. **TESC Only**	<2000 SF new plus replaced hard surfaces	or	<7000 SF land disturbance	MR #2 – Construction Stormwater Pollution Prevention Plan			
2. **New Development** – *All projects*[c]	2000-5000 SF new plus replaced hard surfaces	or	7000-32,670 SF land disturbance	✔		--	--
	>5000 SF new plus replaced hard surfaces	or	>32,670 SF land disturbance	✔	✔	New and replaced hard surfaces	Forested
3a. **Redevelopment** - *Value of proposed improvements is <50% of value of existing site improvements*[c]	2000-5000 SF new plus replaced hard surfaces	or	7000-32,670 SF land disturbance	✔		--	--
	>5000 SF new plus replaced hard surfaces	or	>32,670 SF land disturbance	✔	✔	New hard surfaces only	Forested
3b. **Redevelopment** - *Value of proposed improvements is >50% of value of existing site improvements*[c]	2000-5000 SF new plus replaced hard surfaces	or	7000-32,670 SF land disturbance	✔		--	--
	>5000 SF new plus replaced hard surfaces	or	>32,670 SF land disturbance	✔	✔	New and replaced hard surfaces	Forested
4a. **Transportation redevelopment** - *New hard surfaces add <50% to existing hard surfaces*	2000-5000 SF new plus replaced hard surfaces	or	7000-32,670 SF land disturbance	✔		--	--
	>5000 SF new plus replaced hard surfaces	or	>32,670 SF land disturbance	✔	✔	New hard surfaces only	Forested
4b. **Transportation redevelopment** - *New hard surfaces add >50% to existing hard surfaces*	2000-5000 SF new plus replaced hard surfaces	or	7000-32,670 SF land disturbance	✔		--	--
	>5000 SF new plus replaced hard surfaces	or	>32,670 SF land disturbance	✔	✔	New and replaced hard surfaces	Forested
5. **Central Issaquah Alternative Flow Control Area (see Figure 2-5)** – *All projects*	2000-5000 SF new plus replaced hard surfaces	or	7000-32,670 SF land disturbance	✔		--	--
	>5000 SF new plus replaced hard surfaces	or	>32,670 SF land disturbance	✔	✔	New hard surfaces only	Existing

Figure 1: Project Screening for Stormwater Review

1.1 SUMMARY OF COMPLIANCE ON-SITE

The stormwater design complies with the 9 minimum requirements as follows:

Minimum Requirement #1 – Preparation of Stormwater Site Plans – The Stormwater Site Plan is prepared per the 2014 SWMMWW.

Minimum Requirement #2 – Construction Stormwater Pollution Prevention – A pollution prevention plan will be completed and included with the stormwater site plan as Appendix 7 at the time of the civil permit submittal which will describe the 13 required elements. Further, an erosion control plan will be prepared and included as part of the engineering construction plan set in Appendix 4.

Minimum Requirement #3 – Source Control of Pollution – BMPs listed below are the minimum required for the site, additional BMPs not listed here may need to be implemented the meet the minimum requirements discussed in the 2014 SWMMWW.

- S411 BMPs for Landscaping and Lawn/Vegetation Management
- S417 BMPs for Maintenance of Stormwater Drainage and Treatment Systems
- S421 BMPs for Parking and Storage of Vehicles and Equipment
- S426 BMPs for Spills of Oil and Hazardous Substances

Minimum Requirement #4 – Preservation of Natural Drainage Systems and Outfalls – Currently, stormwater runoff within the parcels sheet flows into the two streams located on and adjacent to the parcels. The roadway frontage along 230th Ave. SE sheet flows into ditches located along the eastern parcel line. A portion of the stormwater runoff from 229th Ave. and 66th Street flows directly into the stream off of the roadway. Ultimately, all of the stormwater runoff is discharged into the streams and taken to the north. After construction, the proposed development will infiltrate 100% of the stormwater runoff on-site. Infiltration systems located under the pavement and in the stream buffer will manage all on-site stormwater runoff from the proposed development. The stormwater runoff from 229th Ave. and 66th Street will no longer sheet flow directly into the stream, it will flow into the proposed gutter and be collected by catch basins. The catch basins will collect the stormwater runoff and convey it into the rain garden facility on-site.

Minimum Requirement #5 – On-site Stormwater Management – In accordance with Minimum Requirement #7, this project is not flow control exempt. Using Table I-2.5.1: On-Site Stormwater Management Requirements for Project Triggering Minimum Requirements #1-9, the proposed project is a new development not located in the UGA on a parcel smaller than 5 acres, therefore the project shall employ the On-Site Stormwater Management BMPs in accordance with the Low Impact Performance Standard or List #2. The project will demonstrate compliance with List #2, see below.

Lawn and Landscaped Areas:

- Per the 2014 SWMMWW manual, BMP T5.13: Post Construction Soil Quality and Depth will be utilized to the maximum extent practicable. See landscape plans for details.

Roofs:

- Full Dispersion (BMP T5.30) or Downspout Full Infiltration Systems (BMP T5.10A): Full dispersion is not feasible for this project site. Full dispersion requires that the site protects at least 65% of the site in a forest or native condition. For this reason alone this BMP is not feasible. In addition, the existing topography and stream locations combined with the site plan does not allow for the required native flow paths at the appropriate slopes (less than 15% away from the target surfaces). Full Infiltration Systems are feasible and will be utilized to the maximum extent practicable. A portion of the stormwater runoff from the roof area will infiltrate in an underground infiltration system. The remaining portion will be infiltrated within a bioretention facility, see below.
- Bioretention (BMP T7.30): Bioretention is feasible for a portion of the proposed project. A bioretention facility will be constructed within 25% of the 75' stream buffer. Stormwater runoff from the a portion of the roof area will be directly tightlined into the bioretention facility.

Other Hard Surfaces:

- Full Dispersion (BMP T5.30): Full dispersion is not feasible for this project site for the reasons mentioned above.
- Permeable Pavement (BMP T5.15): Based on the use of the site and the location of the parcel, both enhanced treatment and phosphorous treatment are required for the stormwater runoff prior to infiltration. A permeable pavement system would not allow for the stormwater runoff to be treated prior to infiltration into the soils.
- Bioretention (BMP T7.30): Bioretention is feasible for a portion of the proposed project. A portion of the stormwater runoff from the frontage improvements and the on-site improvements will be conveyed to a bioretention located within the stream buffer.
- Sheet Flow Dispersion (BMP T5.12) or Concentrated Flow Dispersion (BMP T5.11): Sheet flow dispersion and concentrated flow dispersion are both not feasible for this project. The locations of the existing streams do not allow for the required native flow paths for the stormwater runoff coming off of the target surfaces. Additionally, the requirements that need to be met for Minimum Requirement #6 require that the stormwater runoff be collected and treated prior to infiltration into the soils, this would not be possible prior to dispersion.

Minimum Requirement #6 – Runoff Treatment – The proposed project will construct over 5,000 S.F. of pollution-generating impervious surface, therefore a stormwater treatment facility is required. The SWMMWW states that enhanced treatment is required for project sites that discharge directly to fresh waters or conveyance systems tributary to fresh water designated for aquatic life use or that have an existing aquatic life use; or use infiltration strictly for flow control – not treatment – and the discharge is within ¼ mile of a fresh water designated for aquatic life use. The proposed project will be infiltrated the stormwater runoff within ¼ mile from a fish bearing stream and therefore enhanced treatment is required for all of the pollution-generating impervious surfaces. The proposed project will not be discharging directly into the stream and therefore phosphorous treatment is not required per Section 1.2.2.3 of the 2017 Stormwater Design Manual Addendum. At this time the proposed project is not considered a high-use site, therefore oil-control is also not required. Enhanced treatment for the pollution-generating impervious surfaces will be provided through three Modular-Wetland Systems and the bioretention soil mix located within the rain garden.

Minimum Requirement #7 – Flow Control – The proposed project will construct over 10,000 S.F. of effective impervious surfaces and will not be discharging into flow control exempt waters per Appendix I-E of the SWMMWW, Flow Control-Exempt Surface Waters. Therefore, flow control is required for this project. The proposed project is split into three drainage basins. Two of the drainage basins will provide flow control through infiltration systems comprised of Brentwood Stormcapture Units while the other basin will infiltrate the stormwater runoff in a rain garden facility. 100% of the stormwater runoff from the proposed project and a portion of the frontage improvements will be infiltrated on-site.

Minimum Requirement #8 – Wetlands Protection – There are no wetlands on the project site nor does the project site does currently discharge into a wetland.

Minimum Requirement #9 – Operation and Maintenance – An operations and maintenance manual will be included and attached herein as Appendix 6 at the time of the civil permit submittal.

2. EXISTING CONDITIONS SUMMARY

2.1 EXISTING ON-SITE CONDITIONS

The subject site is +/- 3.92 acres in size. Topography within the property generally flat throughout the site except for the side slopes of the North Fork Issaquah Creek that runs through the northwest corner. In 2017, the Washington State Department of Transportation (WSDOT) conducted the N Fork Issaquah Creek Fish Passage project on this parcel. This project included the following:

- Re-routing the N Fork Issaquah Creek to the west underneath E Lake Sammamish Parkway, instead of straight through the project parcel
- Re-routing a smaller stream to flow directly west under E Lake Sammamish Parkway instead of south under the I-90 off ramp
- Associated improvements to the culverts and downstream flow paths to both streams

Associated with the streams, there are many critical areas on the project site. See Section 2.1.1 of this report for more information. See **Appendix 3** for a preliminary map outlining all the proposed project improvements.

Besides the stream relocation project mentioned above, the site has remained undeveloped since at least 1990. There are no known current drainage flow control facilities on the site. See the figures below.

Figure 2: Existing Conditions (1990)

Figure 3: Existing Conditions (2018)

2.1.1 Flood Hazard Zone

Flood Zones: The project parcel is located with Federal Emergency Management Agency (FEMA) Flood Insurance Rate Map (FIRM) Panel No. 53033C0691H. According to the FIRM Map the project parcel contains Zone AE, Zone AH, and Zone X areas. Zone AE states that base flood elevations have been determined. Zone AH contains flood depths of 1 to 3 feet (usually areas of ponding); base flood elevations determined. The base flood elevation for this specific zone is 72. Zone X includes areas of 0.2% annual chance of flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance of flood. Per Issaquah Municipal Code (IMC) section 16.36.130, the proposed building must be constructed 1 foot above the base flood level. Therefore, the proposed finished floor elevation will be 73. See **Appendix 8** for the FIRM Map.

Critical Area Recharge Area (CARA): According to the Critical Aquifer Recharge Area Classification Map (Exhibit C to Ordinance: CARA Map), the project parcel is located within the Class 1 – 1- & 5-year Wellhead Capture Zone. Per IMC 18.10.796, the City may require a groundwater monitoring plan and/or hydrogeologic critical area assessment report for new development projects. Per IMC 18.06.130, the proposed land use of an Automobile and Truck Sales/Dealership located in an intensive commercial zone and Class 1 CARA is not a prohibited or restricted use (IMC 18.06.130). Groundwater monitoring for the proposed project is in progress.

Streams and Stream Buffers: As mentioned above, the project parcel contains two streams with associated buffers. The N Fork Issaquah Creek is considered a Class 2 stream with salmonids. According to IMC 18.10.780, this stream is smaller than a Class 1 stream that flows year-round during periods of normal rainfall and all streams that are used by salmonids. The smaller stream to the south is considered a Class 4 stream. Per IMC 18.10.785, a Class 4 stream is a constructed or channelized stream, that is intermittent, not used by salmonids and do not provide salmonid habitat, and/or are not directly connected to a Class 1, 2, or 3 stream by an above ground channel. During the WSDOT project mentioned above, the stream buffer was reduced by 25% to create a 75' total buffer width. This buffer width has been added to the proposed project plans. The rain garden facility located within the stream buffer has been designed to have a width of 25% of 75' (18.75') at the maximum water surface elevation. The streams and stream buffers are graphically shown on the exhibit included in **Appendix 3**.

2.1.2 On-Site Soils Information

A geotechnical investigation was conducted by GeoEngineers in November, 2018. Eight test pits and five boring/monitoring wells were conducted to depths of approximately 5 to 81.5 feet. The surficial soils in the vicinity of the site are mapped as alluvial deposits, modified land, recessional outwash and advance outwash. Several stages of outwash and glacial deposition occurred along the Lake Sammamish area and along the outwash channels that carried glacial meltwater into glacial Lake Sammamish. The modified land in this area is typically fill placed to backfill gravel mining activities or to construct embankments for infrastructure. Subsurface soil and groundwater conditions encountered in the explorations were consistent with the geologic mapping. In general, GeoEngineers encountered a surficial layer of fill overlying a relatively thin layer of alluvium which increases in depth to the northwest. Medium dense to dense sand with variable silt, with an occasional layer of gravel with silt and sand underlies the alluvium (recessional outwash potentially transitioning to higher energy glaciofluvial deposits or transitional deposits). Groundwater was encountered during drilling and in the test pit excavations at a depth of 7 to 9 feet in all explorations. Groundwater was measuring as varying between a depth of 6.5 to 8 feet on January 14, 2019. Preliminary infiltration rates varied based on the location of the test pit. The design infiltration rates for the proposed infiltration systems vary based on the locations and the rates given in Table 6 of the geotechnical report. The infiltration systems have been designed to have a minimum separation of 3 feet from the bottom of the facility to groundwater. See **Appendix 5** for the geotechnical report.

3. OFFSITE ANALYSIS REPORT

3.1 Qualitative Upstream Analysis

Currently, stormwater runoff from 66th Street and 229th Avenue sheet flows directly from the roadway and into the stream. Off-site improvements will alter this flow path after construction. The off-site improvements along the south side of 66th Street and 229th Avenue include the construction of a sidewalk, planter strips, curb and gutter, and on-street parallel parking. Currently, the sidewalk ends at the intersection of East Lake Sammamish Parkway and 229th Ave. The proposed project will connect the sidewalk from East Lake Sammamish Parkway to the entrance of the proposed site. The stormwater runoff from the centerline of 229th Avenue and 66th Street currently sheet flows south/southeast directly into the stream buffer and into the stream. After construction, the stormwater runoff will flow along the proposed gutter line, into catch basins and conveyed into the rain garden on-site located in the stream buffer. The stormwater runoff will infiltrate 100% in the rain garden. The frontage improvements to the east of the main entrance on 66th Street will flow along the gutter line around the corner and into 230th Avenue. Stormwater runoff from 230th Avenue currently sheet flows into ditches located on the east and west side of the roadway. After construction of the frontage improvements along 230th Ave., the stormwater runoff from the centerline to the west will be collected by catch basins and conveyed into the ditch to the south as it does today. This outfall will not be altered, and downstream conveyance systems are not anticipated to be adversely affected.

3.2 Qualitative Downstream Analysis

All of the stormwater runoff generated by the disturbed and developed area of the parcel will be infiltrated on-site. There are no anticipated adverse effects to the downstream area of the project site. The proposed frontage improvements along 230th Avenue will not be constructing a significant amount of new impervious surface (<2,000 S.F.) and therefore no adverse effects to the downstream conveyance are anticipated at this time.

4. PERMANENT STORMWATER CONTROL PLAN

4.1 SUMMARY SECTION

The proposed project follows the development requirements stated in the 2014 SWMMWW and the 2017 Addendum to Stormwater Design Manual. Following Figure 2.4.1 (See **Appendix 2**), this project classifies as a new development that triggers all of the minimum requirements. The site does not have 35% or more of existing impervious coverage, and the project will add more than 5,000 S.F. of new impervious surfaces. See **Appendix 4** for the proposed stormwater facility locations and details. Table 1: Land Type Designations Existing vs. Proposed below illustrates the existing and proposed impervious and pervious areas of the disturbed areas (See **Appendix 3** for the basin map).

LAND TYPE DESIGNATIONS	AREA (ACRES)	% OF TOTAL AREA
Existing Areas	**3.58**	**100**
Impervious	0.34	9.50
Pervious	3.24	90.50
Proposed Areas	**3.58**	**100**
Basin 1	**1.86**	**51.96**
Roof	0.03	0.84
Asphalt	1.41	39.39
Sidewalk	0.11	3.07
Landscape	0.31	8.66
Basin 2	**0.34**	**9.50**
Roof	0.13	3.63
Asphalt	0.21	5.87
Sidewalk	0.00	0
Landscape	0.00	0
Basin 3	**1.38**	**38.54**
Roof	0.91	25.42
Asphalt	0.13	3.63
Sidewalk	0.09	2.51
Landscape	0.25	6.98

Table 1: Land Type Designations Existing vs. Proposed

4.1.1 Performance Standards and Goals

Following Figure 2.4.1 – Flow Chart for Determining Requirements for New Development, the project site triggers the use of Minimum Requirements #1-9. All of the stormwater runoff from the disturbed area of the project parcels will be infiltrated on-site. Enhanced treatment will be provided for all of the pollution-generating impervious surfaces through the use of Modular Wetland Systems and infiltration through bioretention soil mix.

4.1.2 Flow Control System

Flow control is required for the proposed development and will be provided through rain gardens, and underground infiltration facilities. The 2012 Western Washington Hydrology Model (WWHM) was used to size the flow control facilities so that they will infiltrate 100% of the stormwater runoff. It is important to note that the underground infiltration facilities will be shallow and maintain a minimum of 3-feet of separation between the bottom of the facility and the groundwater. The drainage plan with the detention/infiltration layouts has been included as **Appendix 4**. See **Appendix 9** for the WWHM reports.

- Basin 1: A 4,375 S.F. x 4-foot-deep infiltration vault consisting of Brentwood Stormtank modules will infiltrate 100% of the stormwater runoff for this basin. Using the information provided by the geotechnical report, an infiltration rate of 5.5 in/hr was used for this facility. This vault will be designed to meet all setback requirements from property lines and structures and will mainly be located within the drive aisle of the parking lot.
- Basin 3: A 1,650 S.F. x 4-foot-deep infiltration vault consisting of Brentwood Stormtank modules will infiltrate 100% of the stormwater runoff from this basin. Using the information provided by the geotechnical report, an infiltration rate of 2 in/hr was used for this facility. This vault will be designed to meet all setback requirements from property lines and structures and will mainly be located within the eastern parking/queuing area.
- Basin 4: A 1.5 foot deep rain garden with a bottom area of 3,760 S.F. will infiltrate 100% of the stormwater runoff from this basin. This rain garden will be located within the stream buffer. It is important to note that the maximum water surface elevation has a width of 18.75’ which is 25% of the 75’ stream buffer.

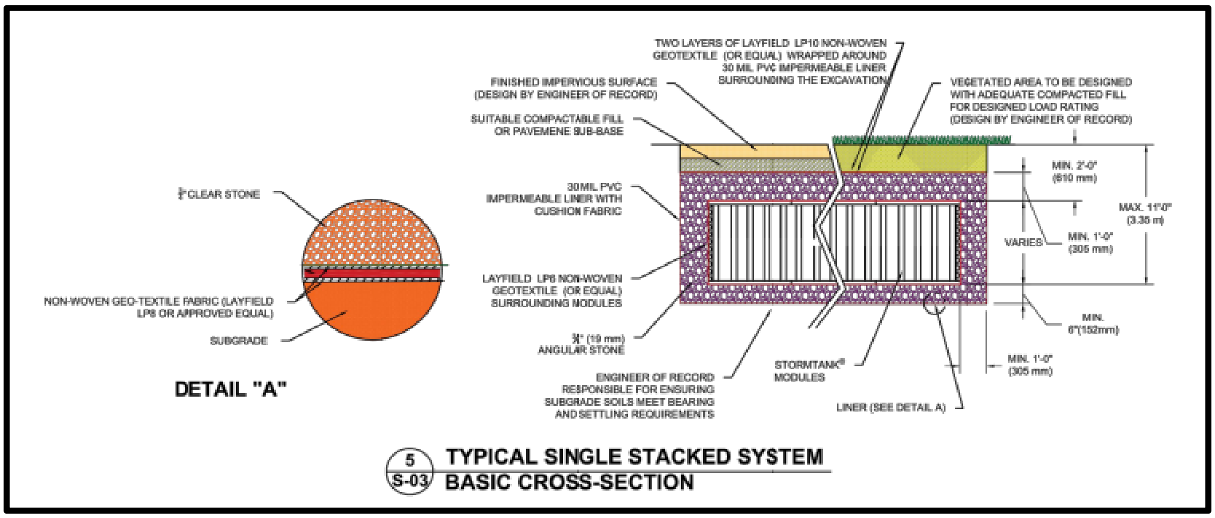


Figure 4: Typical Brentwood Cross Section

4.1.3 Water Quality System

Enhanced treatment will be provided for the proposed development through Modular Wetland Systems and a rain garden. The Modular Wetland Systems will precede the detention/infiltration systems and therefore are required to treat the flow rate at or below which 91% of the runoff volume, as estimated by WWHM. At this stage in design, it is assumed that the stormwater runoff from the sidewalk areas will flow across the asphalt parking areas, and therefore were included in the treatment facility sizing. The Modular Wetland Systems are equipped with an internal bypass and therefore can be sized using the off-line water quality flow rates. See below for the treatment facility sizes. The drainage plan with the locations of the treatment facilities has been included as **Appendix 4**. See **Appendix 9** for the WWHM reports.

- Basin 1:

- All of the basin area was used in treatment sizing, assuming all of the stormwater runoff will flow across the pollution generating impervious surface.
    - Required Water Quality Treatment Flow = 0.1939 cfs (two treatment facilities that will treat 0.097 cfs each)
    - Modular Wetland Size = 8’x8’ and 8’x8’
- Basin 2:
    - 0.21 acres of PGIS (roof area will be directly tightlined to the infiltration facility and therefore does not require treatment)
    - Required Water Quality Treatment Flow = 0.0260 cfs
    - Modular Wetland Size = 4’x4’
- Basin 3: Treatment for this basin will be provided through a rain garden located within the stream buffer. This rain garden has been sized to provide flow control for this basin and will 100% of the stormwater runoff through the bioretention soil mix, and therefore meeting treatment requirements.

4.1.4 Conveyance System Analysis and Design

All stormwater conveyance systems will be sized to convey the 24-hour 25-year storm within the pipe. All proposed stormwater pipes are a minimum of 12” at a minimum slope of 0.5%.

5. CONSTRUCTION STORMWATER POLLUTION PREVENTION PLAN (C-SWPPP)

A SWPPP will be prepared and attached herein as **Appendix 7** at the time of the civil permit submittal.

6. SPECIAL REPORTS AND STUDIES

See **Appendix 5** for the geotechnical report. No other special reports or studies were required for this project.

7. OTHER PERMITS

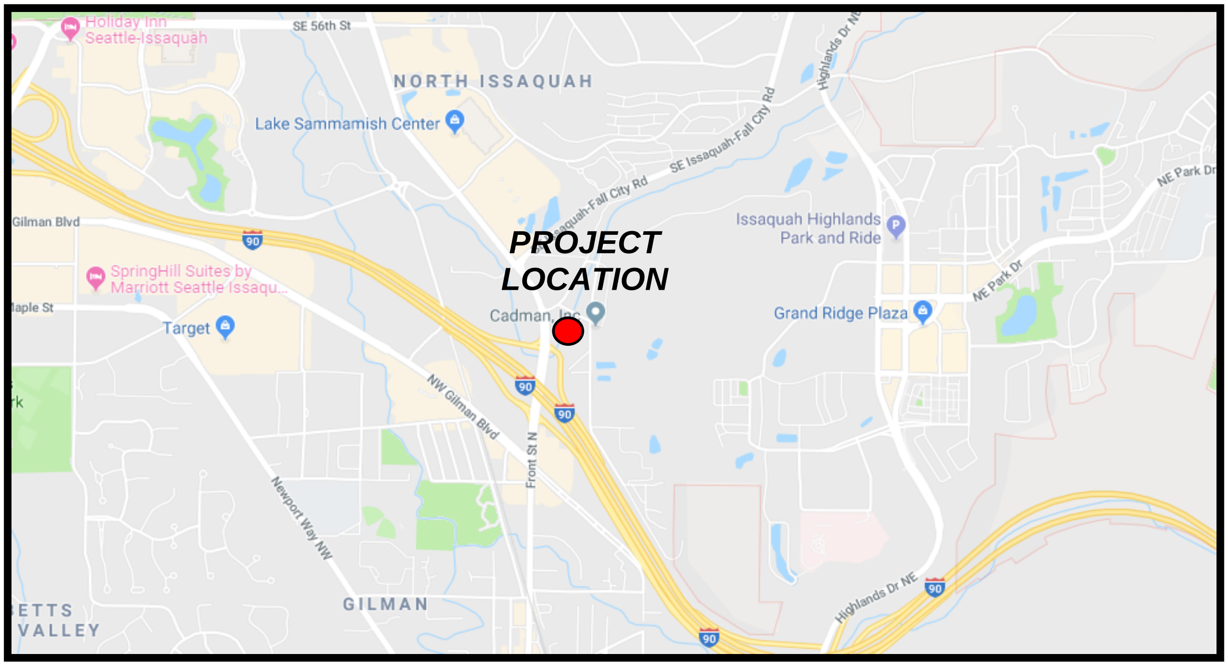
Utility, paving, building, and grading permits may need to be secured prior to beginning construction activities. Coverage under Washington State Department of Ecology Phase II National Pollutant Discharge Elimination System Stormwater Permit will also need to be secured prior to beginning construction activities.

8. OPERATION AND MAINTENANCE MANUAL

The owner of the Evergreen Ford will be responsible in maintaining all stormwater facilities on-site. An operation and maintenance manual will be provided at the time of the civil permit submittal as **Appendix 6**.

END OF STORMWATER SITE PLAN

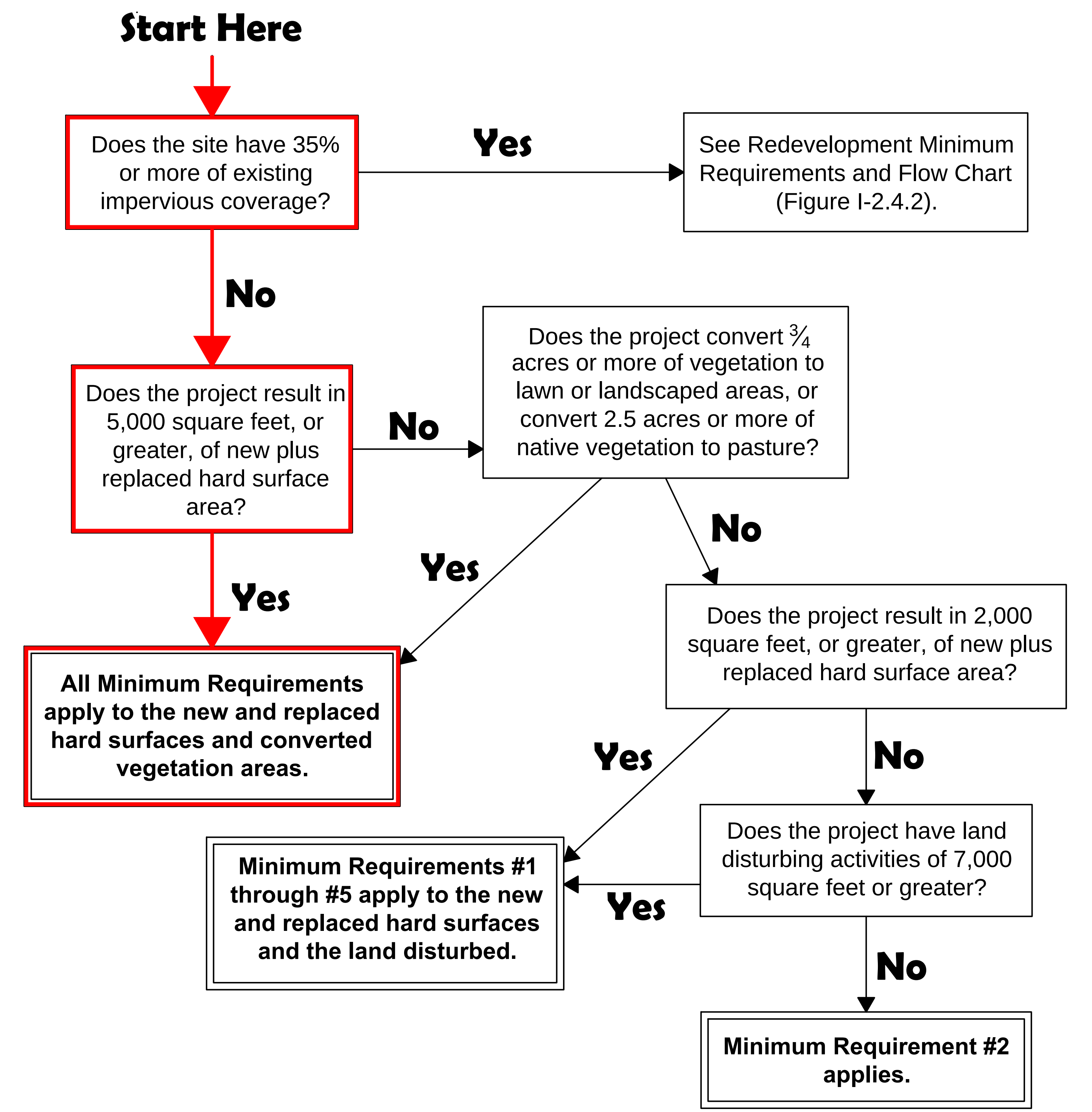
APPENDIX 1
SITE VICINITY MAP


PROJECT
LOCATION
NORTH ISSAQUAH
Lake Sammamish Center
Holiday Inn
Seattle-Issaquah
SE 56th St
Gilman Blvd
SpringHill Suites by
Marriott Seattle Issaqu...
Target
Cadman, Inc
Issaquah Highlands
Park and Ride
Grand Ridge Plaza
SE Issaquah-Fall City Rd
NE Park Dr
Highlands Dr NE
NW Gilman Blvd
Newport Way NW
Front St N
GILMAN
VALLEY
90


PROJECT
LOCATION
Walgreens
SE Issaquah-Fall City Rd
SE 64th Pl
229th Ave SE
SE 66th St
Cadman, In
United States
Postal Service
4th Ave NW
Front St N
NW Gilman Blvd
Salmon Run Nature Park
Pogacha
Issaquah Cannabis
Company
NE Juniper St
NW Juniper St
Car Toys
Berntsen Park
3rd Ct NE
3rd Ave NE
4th Ave NE
5th Pl NE
NE High St
90

APPENDIX 2

DETERMINATION OF MINIMUM REQUIREMENTS WORKSHEET

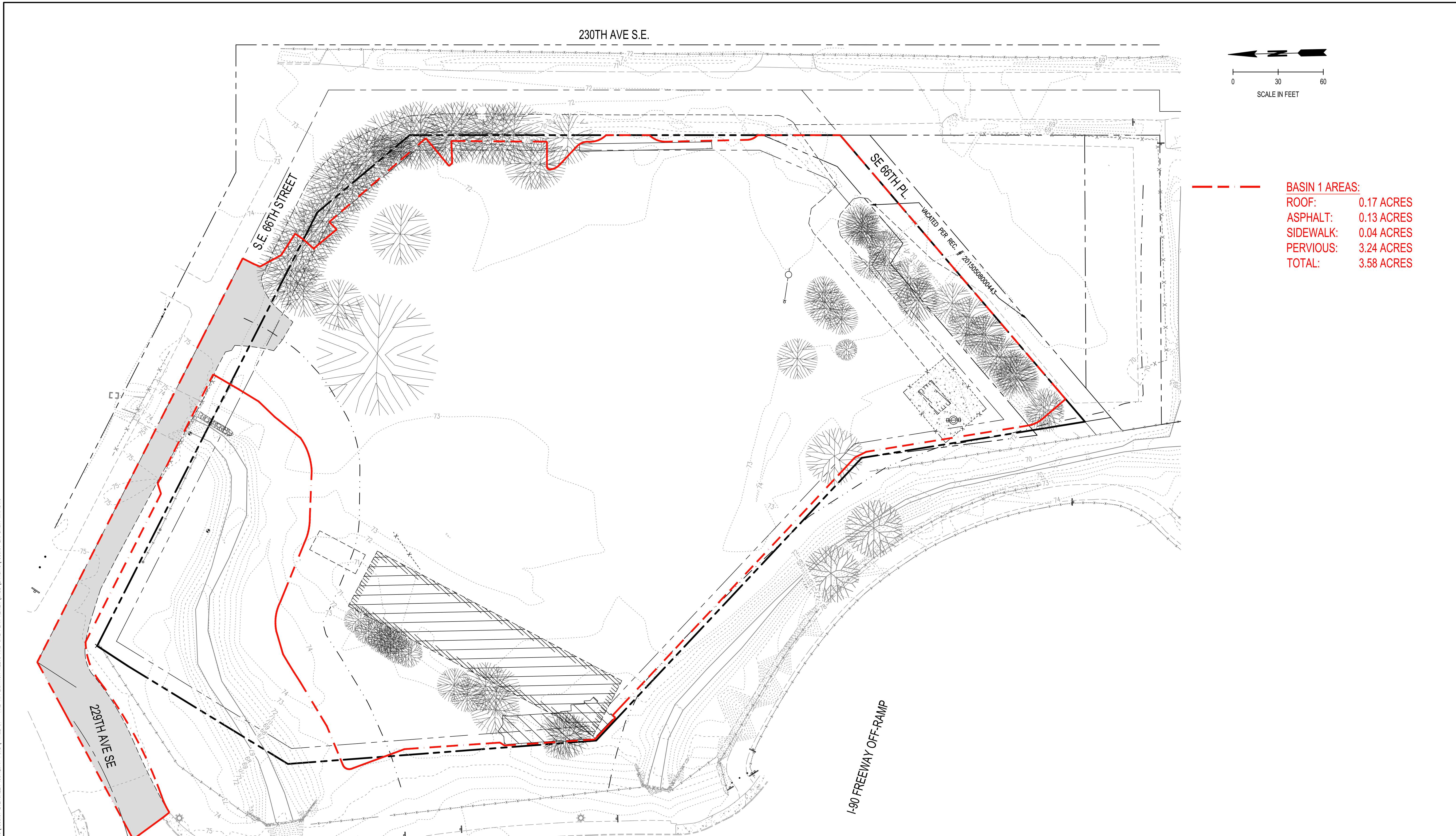


DEPARTMENT OF ECOLOGY
State of Washington

Figure I-2.4.1
Flow Chart for Determining Requirements for New Development

Revised June 2015

APPENDIX 3
BASIN MAP EXHIBITS



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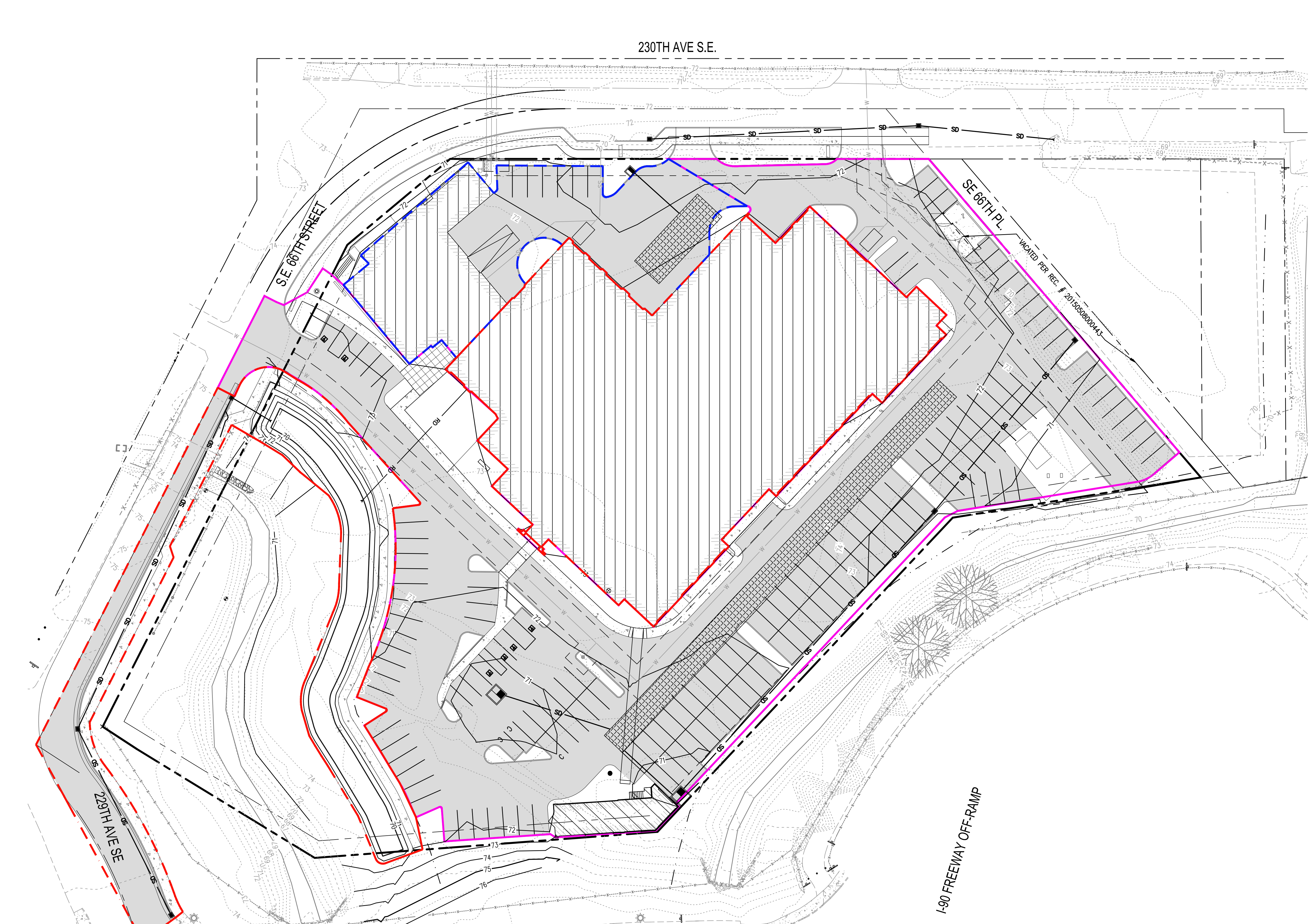
HORIZONTAL SCALE: 1"=30'
DATE: MARCH, 2019
JOB No.: 1883.01
DRAWING FILE No.: 1883.01 Ex. Basin Map.dwg

PROPOSEDE STORMWATER BASIN MAP
ISSAQUAH EVERGREEN FORD

EXHIBIT No: EX-01
SHEET No: 1

Mar 05, 2019 10:32:59am – User mallory.dobbs
N:\PROJECTS\1883 STROTKAMP ARCHITECTS\1883.01 EVERGREEN ISSAQUAH FORD\PHASE 01 – PRE-APPLICATION MEETS AND SITE PLAN REVIEW\CADD\EXHIBITS\1883.01 EX. BASIN MAP.DWG



BASIN 1 AREAS:

ROOF:	0.03 ACRES
ASPHALT:	1.41 ACRES
SIDEWALK:	0.11 ACRES
PERVIOUS:	0.31 ACRES
TOTAL:	1.86 ACRES

BASIN 2 AREAS:

ROOF:	0.13 ACRES
ASPHALT:	0.21 ACRES
SIDEWALK:	0.00 ACRES
PERVIOUS:	0.00 ACRES
TOTAL:	0.34 ACRES

BASIN 3 AREAS:

ROOF:	0.91 ACRES
ASPHALT:	0.13 ACRES
SIDEWALK:	0.09 ACRES
PERVIOUS:	0.25 ACRES
TOTAL:	1.38 ACRES

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CONSULTING SERVICES
8730 TALLON LANE NE, SUITE 200, LACEY, WA 98516
P: 360.352.1465 F: 360.352.1509
SCJALLIANCE.COM

HORIZONTAL SCALE: 1"=30'
DATE: MARCH, 2019
JOB No.: 1883.01
DRAWING FILE No.: 1883.01 Basin Map.dgn

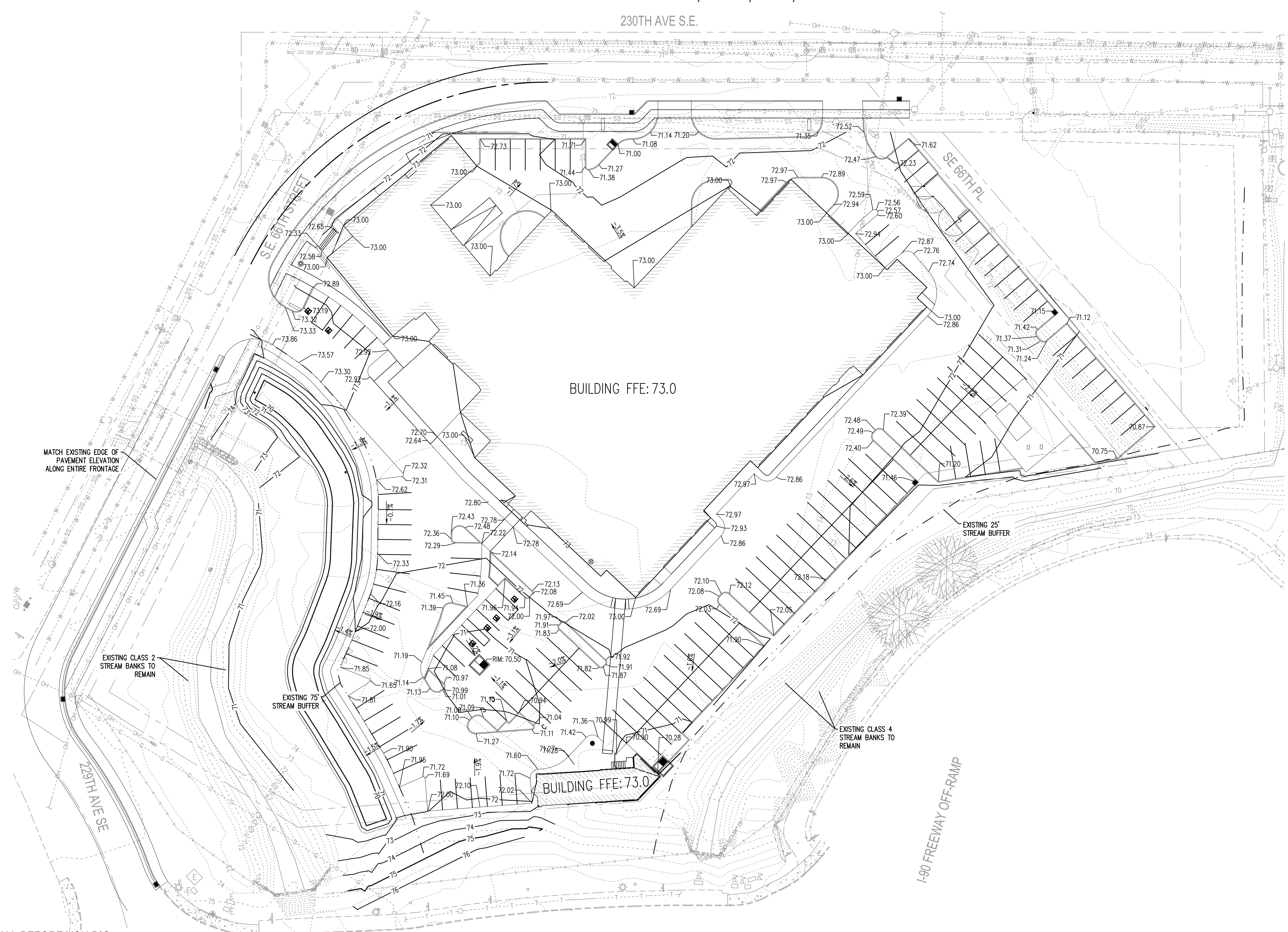
PROPOSEDE STORMWATER BASIN MAP
ISSAQUAH EVERGREEN FORD

EXHIBIT No: EX-02
SHEET No: 2

APPENDIX 4

PRELIMINARY CONSTRUCTION PLANS

SEC. 27, T24N., R6E., W.M.



LEGEND

- XX — EXISTING MAJOR CONTOUR
- XX — EXISTING MINOR CONTOUR
- XX — PROPOSED MAJOR CONTOUR
- XX — PROPOSED MINOR CONTOUR
- GRADE BREAK
- XXX.XX — SPOT ELEVATION
- X% — SLOPE LABEL

NOTES:

FEMA BASE FLOOD ELEVATION: 72.0

CALL BEFORE YOU DIG

THE CONTRACTOR SHALL BE FULLY RESPONSIBLE FOR THE LOCATION AND PROTECTION OF ALL EXISTING UTILITIES. THE CONTRACTOR SHALL VERIFY ALL UTILITY LOCATIONS PRIOR TO CONSTRUCTION BY CALLING THE UNDERGROUND LOCATE LINE AT (800) 424-5555 A MINIMUM OF 48 HOURS PRIOR TO ANY EXCAVATION.

Mar 05, 2019 7:50:07am – User: tyrell.bradley
N:\PROJECTS\1883 STROTKAMP ARCHITECTS\1883.01 EVERGREEN ISSAQUAH FORD\PHASE 01 – PRE-APPLICATION MEETS AND SITE PLAN REVIEW\CADD\1883.01 CG-01.DWG

REVISIONS	DATE	BY

SCJ ALLIANCE
CONSULTING SERVICES
8730 TALLON LANE NE, SUITE 200, LACEY, WA 98516
P: 360.352.1465 F: 360.352.1509
SCJALLIANCE.COM

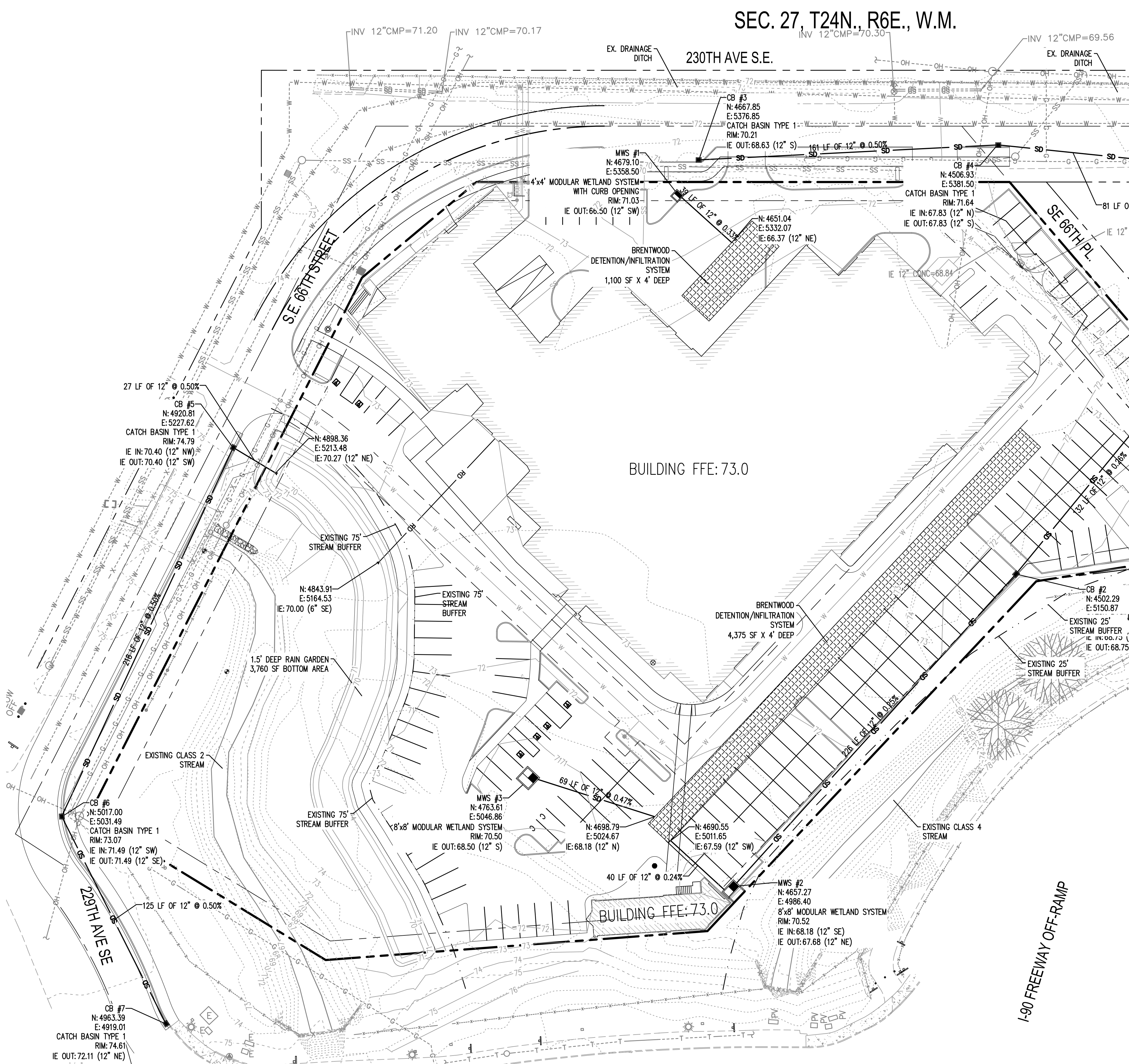
SHEET TITLE: GRADING PLAN

PROJECT NAME: EVERGREEN FORD LINCOLN
230TH AVE SE
ISSAQUAH, WASHINGTON

SEAL: TYRELL E. BRADLEY, STATE OF WASHINGTON, 51055, REGISTERED PROFESSIONAL ENGINEER
03/05/2019

DESIGNER:	M. DOBBS
DRAWN BY:	M. DOBBS
APPROVED BY:	T. BRADLEY
DATE:	MARCH, 2019
JOB NO:	1883.01
DRAWING FILE NO:	1883.01 CG-01
DRAWING NO:	CG-01
SHEET NO:	5 OF 8

SEC. 27, T24N., R6E., W.M.

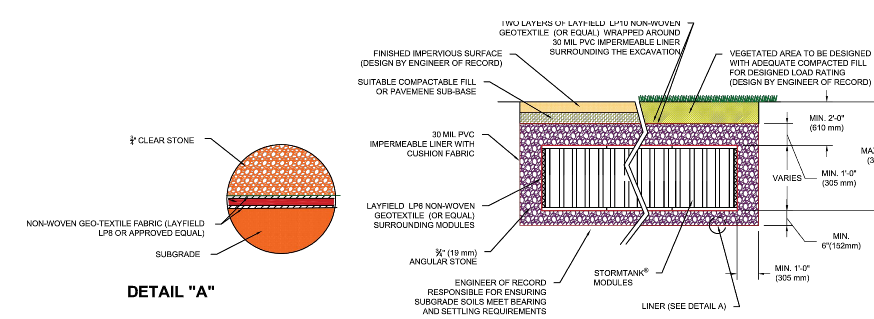


LEGEND

XX	EXISTING CONTOURS (MAJOR/MINOR)
XX	PROPOSED CONTOURS (MAJOR/MINOR)
SD	STORM PIPE
RD	ROOF DRAIN LINE
■	TYPE 1 CATCH BASIN

NOTES:

1. FEMA BASE FLOOD ELEVATION: 72.0
2. SEE STORMWATER SITE PLAN FOR BASIN MAP AND IMPERVIOUS/PERVIOUS AREAS



5 / S-03 **TYPICAL SINGLE STACKED SYSTEM**
BASIC CROSS-SECTION

BRENTWOOD DETENTION/INFILTRATION SYSTEM

NTS

CALL BEFORE YOU DIG

THE CONTRACTOR SHALL BE FULLY RESPONSIBLE FOR THE LOCATION AND PROTECTION OF ALL EXISTING UTILITIES. THE CONTRACTOR SHALL VERIFY ALL UTILITY LOCATIONS PRIOR TO CONSTRUCTION BY CALLING THE UNDERGROUND LOCATE LINE AT (800) 424-5555 A MINIMUM OF 48 HOURS PRIOR TO ANY EXCAVATION.

Mar 05, 2019 7:50:17am – User tyrell.bradley
N:\PROJECTS\1883 STROTKAMP ARCHITECTS\1883.01 EVERGREEN ISSAQUAH FORD\PHASE 01 – PRE-APPLICATION MEETS AND SITE PLAN REVIEW\CADD\1883.01 SD-01.DWG

REVISIONS | DATE | BY

SCJ ALLIANCE
CONSULTING SERVICES
8730 TALLON LANE NE, SUITE 200, LACEY, WA 98516
P: 360.352.1465 F: 360.352.1509
SCJALLIANCE.COM

SHEET TITLE: DRAINAGE PLAN

PROJECT NAME: EVERGREEN FORD LINCOLN
230TH AVE SE
ISSAQUAH, WASHINGTON

SEAL: TYRELL E. BRADLEY, STATE OF WASHINGTON, REGISTERED PROFESSIONAL ENGINEER, 51055, 03/05/2019

DESIGNER:	M. DOBBS
DRAWN BY:	M. DOBBS
APPROVED BY:	T. BRADLEY
DATE:	MARCH, 2019
JOB NO:	1883.01
DRAWING FILE NO:	1883.01 SD-01
DRAWING NO:	SD-01
SHEET NO:	6 OF 8

APPENDIX 5
GEOTECHNICAL REPORT

Geotechnical Engineering Services

Evergreen Ford Lincoln
22909 SE 66th Street
Issaquah, Washington

for
**Strotkamp Associates and
Evergreen Ford Lincoln**

January 18, 2019

GeoEngineers
Earth Science + Technology

Geotechnical Engineering Services

Evergreen Ford Lincoln
22909 SE 66th Street
Issaquah, Washington

for
Strotkamp Associates and
Evergreen Ford Lincoln

January 18, 2019

GeoEngineers
17425 NE Union Hill Road, Suite 250
Redmond, Washington 98052
425.861.6000

Geotechnical Engineering Services

Evergreen Ford Lincoln
22909 SE 66th Street
Issaquah, Washington

File No. 23589-001-00

January 18, 2019

Prepared for:

Evergreen Ford Lincoln
c/o Strotkamp Architects
P.O. Box 501
Burlington, Washington 98233

Attention: Tom Strotkamp and David Estes

Prepared by:

GeoEngineers, Inc.
17425 NE Union Hill Road, Suite 250
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Nancy L. Tochko, PE
Senior Geotechnical Engineer

DEBRA C. OVERBAY
STATE OF WASHINGTON
29247
REGISTERED
PROFESSIONAL ENGINEER
1/18/19

Debra C. Overbay, PE
Associate

DTM:MSH:NLT:DCO:nld

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APPENDICES

INTRODUCTION AND PROJECT UNDERSTANDING

This report presents the results of our geotechnical engineering services in support of the new dealership building and parking lot for Evergreen Ford Lincoln located at 22909 SE 66th Street in Issaquah, Washington. The property is bounded by East Lake Sammamish Parkway SE on the west, 229th Avenue SE and SE 66th Street on the north, 230th Avenue SE on the east, and SE 66th Place and the I-90 off-ramp on the south. The project site is shown relative to surrounding physical features on the Vicinity Map (Figure 1) and Site Plan (Figure 2).

We understand the site is approximately 3½ acres in size, although the northwest corner of the site is occupied by a new creek channel created for the North Fork of Issaquah Creek. The creek channel was originally aligned on the east side of the kennel prior to it being moved in 2017. A second new creek channel borders the southern site boundary along the Issaquah-Preston Trail. The remaining site area is currently vacant, with the exception of an old dog kennel situated in the west and a cell tower located in the south.

We understand that the proposed new dealership will include a four- to five-story, at-grade concrete structure that forms a broad "L" shape measuring approximately 200 feet in length along the northwest and 250 feet along the southwest. The interior dimension will range from about 100 to 150 feet in width. The ground floor will be occupied by sales, service, and associated support facilities and the upper three to four levels will be parking. The development will also include a one-story steel frame building in the north at the intersection of SE 66th Street and 230th Avenue SE, with a walkway to the larger L-shaped concrete structure. The site layout is shown in Figure 2.

Heavy column loads are anticipated due to the concrete structure and upper decks of car loading. Preliminary column loads from PSM Consulting Engineers, the project structural engineer, range from about 350 to 900 kips. We understand the floor load on the ground floor (sales and office) will be on the order of 150 pounds per square foot (psf). Deep foundations will be required to support the structure. The ground floor can be supported at grade provided some damage is acceptable resulting from liquefaction settlement for the design seismic event.

The purpose of this study is to review existing geotechnical information and to complete subsurface explorations at the project site as a basis for providing geotechnical engineering recommendations for design. Our specific scope of services includes:

- reviewing previous explorations completed in the vicinity of the site;
- completing five borings and installing shallow monitoring wells in three of the borings;
- completing eight test pits across the site to better define the characteristics of the near-surface soils and potential compressible deposits;
- providing geotechnical foundation recommendations;
- performing analyses for seismic design, building foundation and floor slab support;
- evaluating infiltration feasibility and provide preliminary infiltration rates based on grain size analyses; and
- preparing this Geotechnical Engineering Design Report.

FIELD EXPLORATIONS

Previous Explorations

GeoEngineers reviewed the logs of explorations completed by others as part of previous studies in the vicinity of the project site. One of the previous borings, B-1, is located on the southern site border from a 1997 project, “Proposed AT&T Tower - Issaquah” by AGRA Earth and Environmental dated February 27, 1997. The location of this boring is shown in Figure 2 and the log is presented in Appendix B, Previous Explorations.

Field Explorations

Subsurface conditions at the site were evaluated by reviewing previous explorations in the immediate vicinity, and by completing eight test pits and five boring/monitoring wells to depths of approximately 5 to 81½ feet below existing ground surface (bgs). The explorations were completed between October 31 and November 2, 2018. The approximate locations of the explorations are shown in Figure 2. A detailed description of the field exploration program and logs of the explorations are presented in Appendix A, Field Explorations and Laboratory Testing.

Laboratory Testing

Soil samples obtained from the test pits and borings were transported to our Redmond geotechnical laboratory and evaluated to confirm or modify field classifications, as well as to evaluate engineering properties of the soil types encountered. Representative samples were selected for laboratory testing consisting of moisture content tests, percent fines, and sieve analyses.

SITE CONDITIONS

Geology

Published geologic information for the project vicinity includes “The Geologic Map of the Issaquah 7.5’ Quadrangle, King County, Washington (Booth, D.B., and Minard, J.P. 1992). The surficial soils in the vicinity of the site are mapped as alluvial deposits, modified land, recessional outwash and advance outwash. Several stages of outwash and glacial deposition occurred along the Lake Sammamish area and along the outwash channels that carried glacial meltwater into glacial Lake Sammamish. Ice contact deposits and transitional deposits are also mapped along the borders of the lake.

The modified land in this area is typically fill placed to backfill gravel mining activities or to construct embankments for infrastructure. Recessional deposits are mapped along the valley wall, below an upland till cap, and below the alluvial deposits.

The alluvial deposits generally consist of interbedded layers of loose/soft soil ranging from sand with variable silt content, to silt and gravel and can contain occasional layers of organic silt/peat. Recessional outwash deposits underlying the alluvium and mapped east of the site mainly consist of medium dense stratified sand and gravel, with some zones of silty sand and silt. Advance glacial and glaciofluvial deposits underlying the recessional outwash deposits mainly consist of dense to very dense sand and gravel with varying amounts of silt.

Surface Conditions

The site is bounded by industrial property to the east, residential and commercial property to the north, by East Lake Sammamish Parkway SE to the west, and by Interstate 90 to the south. The site is relatively flat, with a metal frame deteriorated dog kennel on the east side of the site and a cell tower in the south corner of the site. The site has recently been used for some stockpiled soils, and was recently regraded. Most of the site is covered in newly planted grass and occasional trees. Newly planted landscaped buffers are present along each new stream channel. Above ground high-voltage transmission lines cross the southeast corner of the site near the cell tower.

Subsurface Conditions

Subsurface soil and groundwater conditions encountered in the explorations are consistent with the geologic mapping. In general, we encountered a surficial layer of fill overlying a relatively thin layer of alluvium which increases in depth to the northwest. Medium dense to dense sand with variable silt, with an occasional layer of gravel with silt and sand underlies the alluvium (recessional outwash potentially transitioning to higher energy glaciofluvial deposits or transitional deposits). Soils encountered in our explorations are described in more detail below.

Monitoring wells MW-1 and MW-2 were located in the southeast portion of the proposed building footprint and encountered medium dense silty sand fill with variable gravel content to a depth of about 8 feet below existing site grade. Soft silt and very loose to medium dense silty sand were encountered below the fill. The sand becomes dense to very dense below a depth of about 18 feet. Monitoring well MW-1 encountered dense gravel from a depth of about 19 to 24 feet, and below a depth of 30 feet. The borings were terminated in the sand and gravel at a depth of 31½ feet.

Monitoring well MW-3 and boring B-1 were located toward the northern end of the proposed building. Loose to medium dense silty sand fill was encountered to a depth of about 5 to 8 feet bgs. A layer of loose to very loose silty sand was encountered below the fill in MW-3 to a depth of about 13 feet. Medium dense silty gravel underlies the fill in boring B-1. Loose to dense sand with variable silt and gravel was encountered at depth in both explorations. Dense to very dense sand and gravel deposits were encountered at a depth of 40 to 45 feet. The explorations were terminated in dense sand at a depth between 46 and 52 feet.

Boring B-2 was located in the southeast portion of the proposed building footprint. Approximately 8 feet of loose surficial silty sand fill was also encountered in this boring. Medium dense sand and gravel underlies the fill to a depth of about 18 feet where an approximate 3-foot thickness of organic silt was encountered. Loose to medium dense sand and gravel was encountered below the organic silt to a depth of about 43 feet. An approximate 10-foot layer of dense gravel was encountered between a depth of 43 and 53 feet. Interlayered medium dense to dense sand with variable silt was encountered below this depth to the 80-foot depth explored.

Similar subsurface soil conditions were encountered in the test pits consisting primarily of loose to medium dense silty sand fill with variable gravel. Two test pits, TP-3 and TP-4 encountered a 1- to 1½-foot layer of soft organic silt at a depth of 4 to 5 feet, and test pit TP-7 encountered an approximate 3-foot thickness of soft peat.

Groundwater was encountered during drilling and in the test pit excavations at a depth of 7 to 9 feet in all the explorations. Groundwater was measured as varying between a depth of 6.5 to about 8 feet on

January 14, 2019; the measurements are presented on the respective monitoring well logs. Groundwater conditions should be expected to fluctuate as a function of season, precipitation, and fluctuations of the North Fork Issaquah Creek.

CONCLUSIONS AND RECOMMENDATIONS

We conclude that the site is suitable for constructing the proposed building on deep foundations to support heavy structural loads and to mitigate for settlement due to liquefaction. The site is underlain by liquefiable soils and could experience settlement on the order of 2 to 6 inches during the design seismic event over the majority of the site. Greater liquefaction settlement in the range of 8 to 10 inches is estimated within the west side of the site. Based on the explorations completed to date, deep foundations extending to a depth of 30 to 40 feet in the east, 50 to 55 feet in the north and up to 90 feet in the west corner will support heavy column loads and extend below liquefiable soil layers. Additional explorations should be completed during final design to refine required embedment of deep foundations and confirm liquefiable soil depths.

The first floor should be designed as a structural slab due to the estimated range of liquefaction settlement across the site. Light foundation loads supporting other site facilities can be considered for shallow foundation support provided settlement due to liquefaction is acceptable.

Surficial soils at the site consist mostly of moisture sensitive silty sand fill. Based on the results of the laboratory tests, the on-site soils will likely not be re-usable as structural fill without significant moisture conditioning (aeration). Excavation and replacement of portions of the on-site soils should be anticipated to construct the recommended zone of structural fill beneath the first floor slab and subgrade for the surrounding parking area. Detailed geotechnical recommendations for foundation support and other aspects of project development are presented in the following sections.

Earthquake Engineering

2015 IBC Design Parameters

Based on the subsurface soils encountered in the explorations completed to date, the north and east portions of the proposed building area are underlain by soils classified as Site Class D, and soils encountered in the west corner of the building are classified as Site Class E. We recommend the use of the following 2015 International Building Code (IBC) parameters for soil profile type, short period spectral response acceleration (S_S), 1-second period spectral response acceleration (S_1) and seismic coefficients (F_A and F_V) for the project site.

TABLE 1. 2015 IBC DESIGN PARAMETERS

2015 IBC Parameter	Recommended Value	
	Site Class D	Site Class E
Short Period Spectral Response Acceleration, S_S (percent g)	131	131
1-Second Period Spectral Response Acceleration, S_1 (percent g)	49	49
Seismic Coefficient, F_A	1	0.9
Seismic Coefficient, F_V	1.51	2.4
Peak Ground Acceleration (percent g)	53	48

Surface Faults

The site is more than 2 miles south of the Seattle Fault Zone and therefore it is our opinion the risk of surface fault rupture is low.

Liquefaction

Liquefaction refers to the condition when vibration or shaking of the ground, usually from earthquake forces, results in the development of excess pore pressures in saturated soils with subsequent loss of strength in the deposit of soil so affected. In general, soils that are susceptible to liquefaction include very loose to medium dense clean to silty sands and some silts that are below the water table. Liquefaction usually results in ground settlement and loss of bearing capacity, resulting in settlement of structures that are supported on foundations that are constructed within or above the liquefied soils.

We evaluated the liquefaction potential based on the current and previous explorations using the Simplified Procedure (Youd and Idriss 2001). The Simplified Procedure is based on comparing the cyclic resistance ratio (CRR) of a soil layer (the cyclic shear stress required to cause liquefaction) to the cyclic stress ratio (CSR) induced by an earthquake. The factor of safety against liquefaction is determined by dividing the CRR by the CSR. Liquefaction hazards, including settlement and related effects, were evaluated when the factor of safety against liquefaction was calculated as less than 1.0.

Based on our analysis using the 2015 IBC seismic event (peak horizontal acceleration of 0.53g), it is our opinion there is a moderate to high risk of liquefaction within the upper sand deposits as well as the silt. We estimate that the factor of safety is less than 1.0 during the design-level earthquake for a 10- to 25-foot total thickness of soil in the north, east and south, and up to an approximate 50-foot thickness in the west corner of the proposed building footprint.

The magnitude of liquefaction-induced ground settlement was computed using the Youd and Idriss (2001) simplified approach described previously. Reconsolidation settlement (volumetric strain) is estimated as a function of the factor of safety of liquefaction triggering (serving as a proxy for the maximum accumulated shear strain). Liquefaction-induced ground settlement of the potentially liquefiable zones across the building footprint is estimated to range from 2 inches (in the southeast in the vicinity of monitoring wells MW-1 and MW-2) to as much as 10 inches in the vicinity of boring B-2 for a design-level earthquake. Table 2 below summarizes the range of estimated liquefaction-induced settlement based on the conditions encountered in the explorations.

TABLE 2. ESTIMATED LIQUEFACTION-INDUCED GROUND SETTLEMENT

Boring	Estimated Ground Surface Settlement[1] (inches)
B-1	4 to 6
B-2	8 to 10
MW-1	2
MW-2	2
MW-3	4 to 6

Note:

[1] Additional explorations should be completed during final design to better delineate potential deep liquefaction zones and optimize the building foundation

Lesser amounts of settlement from liquefaction could be experienced after an earthquake with a magnitude less than the design-level earthquake. The magnitude of liquefaction-induced ground settlement will vary as a function of the characteristics of the earthquake (earthquake magnitude, location, duration and intensity) and the soil and groundwater conditions.

Pile Foundations

Based on the presence of potentially liquefiable soils in the upper 20 to 75 feet of the site, and the heavy column loading ranging from about 350 to 900 kips, we recommend that the proposed building be supported on deep foundations. Augercast piles are a common pile foundation in the northwest and typically offer the most economical foundation for heavy column loads. Recommended capacities for 18- and 24-inch-diameter augercast piles are provided below.

Axial Capacity

Table 3 below presents the ultimate pile axial capacities for 18- and 24-inch-diameter piles with a minimum embedment depth of 30 feet in the vicinity of MW-1 and MW-2, 50 feet in the vicinity of boring B-1, and 90 feet at the location of boring B-2. We recommend additional explorations be completed in the footprint prior to contractor bidding to refine the pile depths across the footprint. These ultimate capacities include the down drag force induced by liquefiable soils. A factor of safety of 3 should be used to obtain allowable pile capacities. In addition to the downward compressive load from the seismic structural load, the soil down drag load presented below would need to be included in the structural design analysis.

TABLE 3. ULTIMATE AXIAL PILE CAPACITIES

Pile	Embedment into Dense to Very Dense Sand and Gravel (feet)	Typical Pile Length[1] (feet)			Downward Capacity[2] (kips)	Uplift Capacity (kips)	Down-drag Force (kips)
		MW-1, MW-2	B-1	B-2			
18-inch-diameter augercast	10	30	50	90	390	50	32
	15	35	55	95	425	85	
	20	40	60	100	460	120	
24-inch-diameter augercast	10	--	50	90	740	63	43
	15	35	55	95	790	110	

Notes:

[1] Additional explorations should be completed during final design to refine pile embedment depths.

[2] A factor of safety of 3 should be used to obtain allowable static pile capacity.

Lateral Capacity

Lateral loads can be resisted by passive soil pressure on the vertical piles and by the passive soil pressures on the pile cap. Due to the potential separation between the pile-supported foundation components and the underlying soil from settlement, base friction along the bottom of the pile cap should not be included in calculations for lateral capacity because full contact with the underlying soil cannot be assured.

We completed lateral pile capacity analyses for 18- and 24-inch diameter augercast piles using the computer software program LPILE 2016 produced by Ensoft, Inc. The analyses were completed for both a non-liquefied (static) and liquefied (seismic) soil profile.

Our LPILE analysis results are presented in Figures 3 through 14 as described in the table below. The depths shown on the figures are measured from the bottom of the pile cap; we do not anticipate these results will change significantly with variations in the top of pile elevation.

TABLE 4. LATERAL PILE ANALYSES RESULTS

Figures	Results
Figures 3, 4 and 5	18-inch diameter augercast piles, Boring MW-2 Fixed head deflection, moment and shear diagrams for static and liquefied conditions
Figures 6, 7 and 8	18-inch diameter augercast piles, Boring B-2 Fixed head deflection, moment and shear diagrams for static and liquefied conditions
Figures 9, 10 and 11	24-inch diameter augercast piles, Boring MW-2 Fixed head deflection, moment and shear diagrams for static and liquefied conditions
Figures 12, 13 and 14	24-inch diameter augercast piles, Boring B-2 Fixed head deflection, moment and shear diagrams for static and liquefied conditions

The results presented in Figures 5 through 14 are for single piles. Piles spaced closer than five pile diameters apart will experience group effects that will result in a lower lateral load capacity for trailing rows of piles with respect to leading rows of piles for an equivalent deflection. We recommend that the lateral load capacity for trailing piles in a pile group spaced less than five pile diameters apart be reduced in accordance with the factors in the table below per American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Design Specifications Section 10.7.2.4.

TABLE 5. PILE P-MULTIPLIERS, P_M, FOR MULTIPLE ROW SHADING

Pile Spacing[1] (in terms of pile diameter)	P-Multipliers, P_m[2]		
	Row 1	Row 2	Row 3 and higher[3]
3D	0.8	0.4	0.3
5D	1.0	0.85	0.7

Notes:

[1] The P-multipliers in the table above are a function of the center to center spacing of piles in the group in the direction of loading expressed in multiples of the pile diameter, D.

[2] The values of P_m were developed for vertical piles only.

[3] The P-multipliers are dependent on the pile spacing and the row number in the direction of the loading. To establish values of Pm for other pile spacing values, interpolation between values should be conducted.

Resistance to lateral loads can also be developed by passive pressure on the face of pile caps and other below-grade foundation elements. The allowable passive resistance on the face of grade beams, pile caps, or other embedded foundation elements may be computed using an equivalent fluid density of 250 pounds per cubic foot (pcf) (triangular distribution) if these elements are cast in direct contact with undisturbed on-site soils. Alternatively, passive pressures may be computed using an equivalent fluid density of 350 pcf if all soil extending out from the face of the foundation element for a distance at least equal to two and one-half times the depth of the element consists of structural fill compacted to at least 95 percent of maximum dry density (MDD) (ASTM D-1557). This passive resistance value includes a factor of safety of

1.5 and a minimum lateral deflection of 1 inch to fully develop the passive resistance. Deflections less than 1 inch will not fully mobilize the passive resistance and can be linearly interpolated from the resistance at 1 inch.

Pile Installation

Augercast piles should be installed to the recommended penetrations using a continuous-flight, hollow-stem auger. The pile grout is pumped under pressure through the hollow stem as the auger is slowly withdrawn. Reinforcing steel for bending and uplift is placed in the fresh grout column immediately after withdrawal of the auger.

We recommend that the augercast piles be installed by a contractor experienced in their placement and using suitable equipment. Grout pumps should be fitted with a volume-measuring device and pressure gauge so that the volume of grout placed in each pile and the pressure head can be readily determined. While grouting, the rate of auger withdrawal should be controlled such that the rate is uniform and the volume of grout pumped is equivalent to at least 115 percent of the theoretical hole volume. A minimum grout line pressure of 100 pounds per square inch (psi) should be maintained while grouting. We recommend that there be a waiting period of at least eight hours between installation of piles spaced closer than 8 feet center-to-center, in order to avoid disturbance of concrete undergoing curing in a previously cast pile. This is particularly important for the anticipated depth and the loose soil consistency at the site. These materials can sometimes experience a “blow out” from grout pressures during augercast pile installation.

It should be noted that the recommended pile tip elevations and capacities presented above are based on assumed uniformity of soil conditions between the explorations. Obstructions could be encountered within the fill soils during installation such that new pile locations may need to be selected and/or pile capacities may need to be reevaluated. There may be unexpected variations in the depth to, and characteristics of, the supporting soils across the site. In addition, no direct information regarding the capacity of augercast piles (e.g., driving resistance data) is obtained while this type of pile is being installed. Therefore, it is particularly important that the installation of augercast piles be carefully monitored by a representative from our firm who will work under the direct supervision of an experienced engineer familiar with the conditions at this site.

Floor Slab

As discussed previously, we estimate that potential settlements from soil liquefaction during a design earthquake event could vary significantly across the site. Because the estimated settlements are not tolerable, a structural floor slab is recommended. We recommend the floor slab be underlain by a minimum 4-inch-thick capillary break layer. to provide uniform support and drainage. Gradation recommendations for the capillary break are presented in the “Earthwork” section below.

If water vapor migration through the slabs is objectionable, such as in occupied spaces or areas where adhesives are used to anchor carpet or tile to the slab, the capillary break material should be covered with a commercial moisture vapor retarder (10-mil minimum thickness with lapped and sealed seams). The moisture vapor retarder should be constructed in accordance with the American Concrete Institute (ACI 302.1R) and placed over the capillary break layer. The contractor should be made responsible for maintaining the integrity of the vapor barrier during construction.

A waterproofing product designed for this purpose may be used in lieu of the capillary break material and vapor retarder if a more robust level of protection is desired.

Ground Improvement

Methods and Design Considerations

Ground improvement can be considered to mitigate liquefaction and provide increased bearing pressures for shallow footings. However, the depth and thickness of liquefiable soils vary significantly across the building footprint, and extend up to a depth of about 75 feet in boring B-2. Additional explorations should be completed to verify the extent and thickness of liquefiable zones during final design. The ground improvement system should be designed so that abrupt differential settlements do not occur along the transition line between differing thicknesses of liquefiable soil, and between improved ground and non-improved ground. As such, ground improvement may not be practical in areas of deep liquefiable soils.

Ground improvement options may include rigid inclusions, aggregate piers, and driven timber piles to mitigate liquefication and provide increased bearing for shallow foundations. The ground improvement elements should be installed in a grid pattern beneath footings, and also at regular intervals beneath the ground floor slab, as needed to limit slab settlements.

Rigid inclusions are unreinforced low strength concrete elements that transfer foundation loads through weak soils down to underlying competent soils. These are typically installed using a bottom-feed mandrel that is vibrated down to the bearing soils. Granular bearing soils are densified by displacement. Low strength concrete is pumped through the mandrel, which opens at the bottom as it is raised. The mandrel is extracted while a positive concrete pressure is maintained.

Rammed aggregate piers consist of holes created by driving/vibrating a mandrel which are then filled with densely compacted crushed rock. The holes are advanced down to suitable bearing soils. The crushed rock is placed in the hole in lifts of about 12 inches in thickness as the mandrel is withdrawn and compacted using a high energy hydraulic ram. Grout can be added to the portion of the crushed rock column extending through the peat in order to provide higher lateral stiffness and therefore a higher vertical load capacity and smaller foundation settlements.

Each of these methods involve displacing rather than replacing the existing soil. Accordingly, the resulting composite soil mass has improved strength, lower compressibility, and low liquefaction potential. Also, foundation loads are transferred to the underlying competent bearing soils.

The ground improvement systems would be completed on a grid pattern, where necessary, to transfer the foundation loading to the bearing soils. The type of ground improvement technique should be reviewed with the project team to identify constructability issues, provide a range of cost, and to establish the allowable bearing that can be achieved using the method selected.

A contractor specializing in ground improvement methods should develop a performance-based design that will meet the support and settlement criteria specified by the project structural engineer. We recommend that we be retained to review the proposed ground improvement program.

Construction Considerations

Installation of ground improvement elements may encounter seepage or heaving conditions due to the high groundwater levels present at the site, and zones of medium dense to dense gravel. Measures should be taken to prevent sloughing, caving, heaving or running of soil into the holes. Casing or other techniques may be necessary to stabilize the holes. Also, concrete and asphalt pieces, debris, cobbles or boulders may be encountered during installation of the ground improvement elements.

Each of the ground improvement methods will generate vibrations during installation. These vibrations are not expected to adversely affect nearby off-site structures. However, it is likely that the vibrations will be felt by people within a limited area in and adjacent to the site.

GeoEngineers should observe and document the installation of the selected ground improvement method to verify conformance with the design assumptions and recommendations.

In our experience, building foundations bearing on a crushed rock pad overlying improved ground are typically designed using an allowable bearing pressure up to 5,000 psf.

Shallow Foundation Support

At this time, it is unknown whether there might be small retaining walls or other small structures supported at grade. If small non-pile-supported structures are planned, we recommend that footings be founded on at least 2 feet of structural fill. The zone of structural fill should extend laterally beyond the footing edges a horizontal distance at least equal to the thickness of the fill. An allowable soil bearing pressure of 2,500 psf may be used for the footings, provided that the foundations have a minimum width of 2 feet and bear on a minimum of 2 feet of compacted structural fill. These bearing pressures apply to the sum of all dead plus long-term live loads, excluding the weight of the footing and any overlying backfill. These values may be increased by one-third when wind or seismic loads are considered. Foundation settlement for these support conditions under static loads is estimated to be on the order of ½ to 1 inch. This type of support might result in significant settlement if liquefaction of underlying soils occurs during an earthquake. Foundation settlements if liquefaction occurs could be on the order of 2 to 10 inches, as discussed previously.

We recommend a minimum embedment of 18 inches for shallow foundations for frost depth. As the structural fill will be founded on undocumented existing fill, we strongly recommend that all prepared foundation subgrades be observed by a representative of GeoEngineers to confirm that unsuitable fill (for example, fill containing trash or significant organics/wood debris) is not present.

Retaining Walls

We recommend that walls for loading docks or other building walls which will serve as retaining walls be designed for lateral pressures based on an equivalent fluid density of 35 pcf. This assumes that the walls will not be restrained against rotation when backfill is placed. The above-recommended lateral soil pressure does not include the effects of surcharges such as floor loads, traffic loads or other surface loading. Surcharge effects should be considered as appropriate.

In settlement-sensitive areas (e.g., beneath on-grade slabs), the upper 2 feet of backfill for subgrade walls should be compacted to at least 95 percent of the MDD determined in accordance with ASTM D-1557. At other locations and below a depth of 2 feet, wall backfill should be compacted to between 90 and 92 percent of ASTM D-1557. Measures should be taken to prevent overcompaction of the backfill behind the wall. This can be achieved by placing the zone of backfill located within 5 feet of the wall in lifts not exceeding 6 inches in loose thickness and compacting this zone with hand-operated equipment such as a vibrating plate compactor.

The recommended equivalent fluid density assumes a free-draining condition behind the wall. This may be achieved by placing an 18- to 24-inch-wide zone of sand and gravel containing less than 5 percent fines against the wall. Weep holes at about 4-foot centers at the base of the wall should be sufficient to drain water from exterior walls. Alternatively, perforated drainpipe could be embedded in the free-draining sand and gravel zone along the base of retaining walls to remove any water which collects in this zone. The drainpipe should be tightlined to an appropriate discharge point.

Lateral Resistance

The soil resistance available to resist lateral loads is a function of the frictional resistance which can develop on the base of footings and floor slab, and the passive resistance which can develop on the face of below-grade elements of the structure as these elements tend to move into the soil. For footings and floor slabs founded on structural fill placed and compacted in accordance with our recommendations, the allowable frictional resistance may be computed using a coefficient of friction of 0.35 applied to vertical dead-load forces. The allowable passive resistance previously recommended in the Pile Foundations section is appropriate for retaining wall design.

Earthwork

Subgrade Preparation

The exposed subgrade should be evaluated after grading is complete and prior to placing base course by proof-rolling with a loaded dump truck. The proof-roll should be observed by a representative from our firm to confirm the subgrade performance. The exposed soil should be firm and unyielding, and without significant groundwater.

If the exposed subgrade is not acceptable based on the proof-roll, we recommend that unsuitable soils be overexcavated to a maximum depth of 2 feet and replaced with imported structural fill.

Structural Fill

Materials used as fill at the site should meet the requirements below.

- Structural fill placed to support foundations, slabs-on-grade, or driveway, parking and sidewalk areas should meet the requirements of gravel borrow, Washington State Department of Transportation (WSDOT) gravel borrow, WSDOT Standard Specification 9-03.14(1).
- We recommend that structural fill placed for wall or footing drainage systems consist of WSDOT gravel backfill for drains, WSDOT Standard Specification 9-03.12(4).
- Structural fill placed as capillary break material below the floor slab should meet the requirements of WSDOT Standard Specification 9-03.1(4)C, grading No. 57 (1-inch minus crushed rock).

Structural fill must be mechanically compacted to a firm, non-yielding condition. Structural fill must be placed in loose lifts not exceeding 12 inches in thickness. Each lift must be conditioned to the proper moisture content and compacted to the specified density before placing subsequent lifts. Structural fill must be compacted to the following criteria:

- Structural fill placed to support foundations, slab-on-grade, or driveway, parking and sidewalk areas should be compacted to at least 95 percent of the MDD per ASTM International (ASTM) D 1557.
- Structural fill placed to backfill utility trenches should be compacted to between 90 and 92 percent of the MDD per ASTM D 1557, except for the upper 2 feet that should be compacted to at least 95 percent of MDD.

We recommend that GeoEngineers be present during proof-rolling and/or probing of the exposed subgrade soils in pavement areas, and during placement of structural fill. We will evaluate the adequacy of the subgrade soils and identify areas needing further work, perform in-place moisture-density tests in the fill to verify compliance with the compaction specifications, and advise on any modifications to the procedures that may be appropriate for the prevailing conditions.

Reuse of On-site Soils

The on-site soils that will be excavated for construction of the building slab, pile caps, utilities and pavement contain a high percentage of fines; we anticipate that most of the excavated soils will be moisture-sensitive and only be suitable for use in landscaping areas and will not be suitable for reuse as structural fill. On-site soils reused in landscaping areas will likely need amendment to meet landscaping requirements.

If augercast piles are selected as the preferred foundation system, the spoils from construction of the piles will be wet and will need to be disposed of off-site. The spoils will be a mixture of the upper sand, silt, and pile grout and will not be suitable for landscaping areas.

Temporary Excavations

We anticipate that most excavations required for the project will be relatively shallow, on the order of 4 to 6 feet in depth for the pile caps and utilities. We anticipate that the depth of the excavations required for the pile caps will generally be above the water table. Groundwater may be encountered above this depth if work takes place during or immediately after extended wet weather. We anticipate that the groundwater can be handled during construction by pumping from sumps, as necessary. All collected water should be routed to suitable discharge points.

Excavations deeper than 7 to 8 feet below existing site grades will likely encounter groundwater that will be difficult to handle by sumps alone. A dewatering plan should be developed by the contractor for excavations deeper than about 7 feet.

Temporary Cut Slopes

All temporary cut slopes and shoring must comply with the provisions of Title 296 Washington Administrative Code (WAC), Part N, “Excavation, Trenching and Shoring.” The contractor performing the work has the primary responsibility for protection of workers and adjacent improvements.

We recommend temporary cut slope inclinations of 1½H:1V (horizontal to vertical) in the soils encountered at the site. Some caving/sloughing of the cut slopes may occur at this inclination. The inclination may need to be flattened by the contractor if significant caving/sloughing occurs. These cut slope recommendations apply to fully dewatered conditions. For open cuts at the site, we recommend that:

- no traffic, construction equipment, stockpiles or building supplies be allowed at the top of the cut slopes within a distance of at least 5 feet from the top of the cut;
- exposed soil along the slope be protected from surface erosion using waterproof tarps, plastic sheeting or flashcoating with shotcrete;
- construction activities be scheduled so that the length of time the temporary cut is left open is reduced to the extent practicable;
- erosion control measures be implemented as appropriate such that runoff from the site is reduced to the extent practicable;
- surface water be diverted away from the excavation; and
- the general condition of the slopes should be observed periodically by GeoEngineers to confirm adequate stability.

Because the contractor has control of the construction operations, the contractor should be made responsible for the stability of cut slopes, as well as the safety of the excavations. The contractor should take all necessary steps to ensure the safety of the workers near slopes.

Temporary Shoring

Because of the diversity of available shoring systems and construction techniques, the design of temporary shoring is most appropriately left up to the contractor proposing to complete the installation. The following paragraphs present recommendations for the type of shoring systems and design parameters that we conclude are appropriate for the subsurface conditions at the site.

The soils within the project area can be retained using conventional trench shoring systems such as trench boxes, sheet piles, a braced system, or a slide rail system. The design of temporary shoring should allow for lateral pressures exerted by the adjacent soil, surcharge loads from traffic, construction equipment and temporary stockpiles adjacent to the excavation, etc.

The lateral soil pressures acting on temporary shoring will depend on the nature and density of the soil behind the wall, the inclination of the ground surface behind the wall, and the groundwater level. For walls that are free to yield at the top at least one thousandth of the height of the wall (i.e., wall height times 0.001), soil pressures will be less than if movement is restrained. Lateral load resistance can be mobilized through the use of braces, tiebacks, anchor blocks and passive pressures on members that extend below the bottom of the excavation. Temporary shoring used to support trench excavations typically uses internal bracing such as hydraulic shoring or trench boxes.

We recommend that yielding walls retaining the existing soils be designed using an equivalent fluid density of 40 pcf, for horizontal ground surfaces. For non-yielding (i.e., braced) systems, we recommend that the shoring be designed for a uniform lateral pressure of 26H in psf, where H is the depth of the planned excavation in feet below a level ground surface. These values assume that the ground behind the shoring has been dewatered such that the ground water table is at least 2 feet below the base of the excavation.

If the dewatering system is not designed to lower the groundwater level behind the shoring walls (e.g. sheet pile walls with dewatering system inside the shored excavation), hydrostatic pressures must be included in the shoring design. For this condition, temporary shoring should be designed using a lateral pressure equal to an equivalent fluid density of 85 pcf, for horizontal ground conditions adjacent to the excavation.

The above lateral soil pressures do not include traffic, structure or construction surcharges that should be added separately, if appropriate.

The soil pressure available to resist lateral loads against shoring is a function of the passive resistance that can develop on the face of below-grade elements of the shoring as those elements move horizontally into the soil. The allowable passive resistance on the face of embedded shoring elements may be computed using an equivalent fluid density of 125 pcf. This passive equivalent fluid density value is for soil below the water table and includes a factor of safety of about 1.5.

Weather Considerations

The on-site soils generally contain a sufficiently high percentage of fines (silt and clay) and are therefore moisture-sensitive. When the moisture content of these soils is more than a few percent above the optimum moisture content, these soils become muddy and unstable, operation of equipment on these soils will be difficult, and it will be difficult or impossible to meet the required compaction criteria. Additionally, disturbance of near-surface soils should be expected if earthwork is completed during periods of wet weather. It will be preferable to schedule site preparation and earthwork activities during extended periods of dry weather when the soils will: (1) be less susceptible to disturbance; (2) provide better support for construction equipment; and (3) be more likely to meet the required compaction criteria.

The wet weather season in western Washington generally begins in October and continues through May; however, periods of wet weather may occur during any month of the year. The optimum earthwork period for these types of soils is typically June through September. If wet weather earthwork is unavoidable, we recommend the following:

- The ground surface in and around the work area should be sloped so that surface water is directed away from the work area. The ground surface should be graded such that areas of ponded water do not develop. The contractor should take measures to prevent surface water from collecting in excavations and trenches. Measures should be implemented to remove surface water from the work area.
- Slopes with exposed soils should be covered with plastic sheeting or similar means.
- The site soils should not be left uncompacted and exposed to moisture. Sealing the surficial soils by rolling with a smooth-drum roller prior to periods of precipitation will reduce the extent to which these soils become wet or unstable.
- Construction traffic should be restricted to specific areas of the site with appropriate best management practices (BMPs) to control sedimentation.
- Construction activities should be scheduled so that the length of time that soils are left exposed to moisture is reduced to the extent practicable.

Preliminary Infiltration Considerations

We understand infiltration facilities are being considered in the proposed parking areas. Preliminary infiltration rates were estimated based on grain size analyses using the guidelines in the Stormwater Management Manual for Western Washington (SMMWW) adopted by City of Issaquah. The preliminary rates should be confirmed by pilot infiltration testing when the facility location and depth is determined. Based on our experience, the rates calculated by the grain size method are typically higher than in-situ measurements.

TABLE 6. PRELIMINARY INFILTRATION RATES

Exploration	Soil Type	Depth (feet)	Short-Term Infiltration Rate (in/hr) uncorrected	Correction Factor CF[1]	Estimated Design (Long-term) Infiltration Rate (in/hr)
TP-2	Gravel with Silt and Sand	5	19.7	0.119	2.34
TP-4	Silty Sand	1	9.7	0.119	1.15
TP-4	Sand with Silt	5	52.5	0.119	6.24
TP-5	Gravel with Silt and Sand	2	31.0	0.119	3.69
TP-8	Silty Sand	2	12.0	0.119	1.43
MW-2	Sand with Silt	5	46.4	0.119	5.51
MW-3	Silty Sand	5	5.7	0.119	0.68

Notes:

[1] Total Correction Factor based on CF_v = 0.33, Cf_t = 0.4 and long-term conductivity loss factor = 0.9

Utilities

Trench excavation, pipe bedding, and trench backfilling should be completed using the general procedures described in the WSDOT Standard Specifications or other suitable procedures specified by the project civil engineer. Utility pipes should be bedded with bedding material as specified by the project civil engineer. We recommend a minimum 6-inch-thick layer, or one-fourth of the pipe diameter, whichever is greater, of pipe bedding material be placed below, above, and around the perimeter of the pipe. This bedding material should be lightly tamped into place. Backfill placed above the bedding material shall consist of structural fill quality material as discussed above.

Utility trench backfill should be placed in lifts of 12 inches or less (loose thickness) such that adequate compaction can be achieved throughout the entire lift. Each lift must be compacted prior to placing the subsequent lift. Prior to compaction, the backfill should be moisture conditioned to near optimum moisture content, if necessary. The backfill should be compacted in accordance with the criteria discussed above.

Dewatering

We recommend that the ground water level be maintained 1 to 2 feet below the bottom of excavations during construction, or that level necessary to stabilize the shoring and provide a firm subgrade. Quarry spalls and pea gravel can be used as bedding for utilities that extend below the groundwater level. The groundwater level will depend upon the dewatering method, the size of the excavation and other factors. We do not anticipate that a significant dewatering effort will be required during construction of shallow utilities. However, vaults or tanks extending below the groundwater level may be required, in which case more extensive dewatering will be necessary (well points or deep wells). Any seepage that enters the shallow utility excavations can likely be handled by the use of sumps and pumps.

Buoyancy

The effects of buoyancy should be considered in design of the utilities and vaults extending deeper than 8 feet bgs. Buoyancy effects can be resisted by the dead weight of the structure, friction along the sides of the structure, and the weight of zones of soil which are located above the slab floor which protrude beyond the permanent walls. Frictional resistance can be computed using a coefficient of friction of 0.4 applied to the lateral soil pressures. This coefficient of friction value includes a factor of safety of about 1.5. We recommend that lateral soil pressure for uplift resistance be computed using an equivalent fluid density of 20 pcf considering groundwater is present. Backfill above the slab floor may be assumed to have a submerged unit weight of 57 pcf.

Pavement Recommendations

Subgrade Preparation

Pavement subgrade areas should be stripped and proofrolled, or probed to evaluate the existing subgrade surface prior to placing new fill for pavement support or the new pavement section. Where the existing soils are loose or wet and cannot be compacted, it will be necessary to excavate and replace these soils. The required excavation thickness will depend on the moisture content of the subgrade soils at the time of construction and should be evaluated at that time. To avoid the cost of additional overexcavation, the pavement subgrade preparation should occur during the dry season as practical.

Design Section

Based on our experience with similar developments, we recommend the following minimum pavement design sections. The heavier section should be utilized throughout the site if automobile parking areas cannot be strictly designated.

TABLE 7. RECOMMENDED DESIGN PAVEMENT SECTIONS

Pavement Area	HMA CL. ½ PG 64-22[1] (inches)	Crushed Surfacing Base Course with less than 5 percent fines content[2] (inches)
Automobile Parking	2	6
Entrance Drive and Heavier Truck Traffic	3	6

Notes:

[1] Hot mix asphalt (HMA) Class ½-inch, PG 64-22 per WSDOT Standard Specification 5-04 and 9-03. Minimum 2-inch thickness recommended.

[2] Crushed Surfacing per WSDOT Standard Specification 9-03.9(3) compacted to 95 percent of the MDD determined using ASTM D-1557, to contain less than 5 percent fines content and to be placed on subgrade compacted to 95 percent of MDD.

Drainage Considerations

We recommend that pavement surfaces be sloped so that surface drainage flows away from the building, and all roof drainage be collected in tight lines for diversion into the storm drain system. A perimeter footing drain is recommended to intercept surface water runoff that may be perched on the surficial silty sand soils. All areas should be graded to avoid concentration of runoff onto fill or cut slopes or other erosion-sensitive areas.

Recommended Additional Geotechnical Services

Throughout this report, recommendations are provided where we consider additional geotechnical services to be appropriate. These additional services are summarized below:

- Additional insitu testing (pilot infiltration testing) should be completed at site specific infiltration facilities to confirm infiltration rates.
- If augercast piles are selected as the foundation system, we recommend additional explorations be completed to refine the required pile embedment depths across the footprint.
- GeoEngineers should be retained to review the project plans and specifications when complete to confirm that our design recommendations have been implemented as intended.
- During construction, GeoEngineers should observe and document installation of deep foundations or ground improvement, evaluate the suitability of the pavement and slab subgrades, observe and test structural backfill and provide a summary letter of our construction observation services. The purposes of GeoEngineers construction phase services are to confirm that the subsurface conditions are consistent with those observed in the explorations and other reasons described in Appendix C, Report Limitations and Guidelines for Use.

LIMITATIONS

We have prepared this report for the exclusive use of Strotkamp Associates. and members of the design team for the Evergreen Ford Lincoln property in Issaquah, Washington. Our report, conclusions and interpretations should not be construed as a warranty of the subsurface conditions.

Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices in the field of geotechnical engineering in this area at the time this report was prepared. No warranty or other conditions, express or implied, should be understood.

Any electronic form, facsimile or hard copy of the original document (email, text, table, and/or figure), if provided, and any attachments are only a copy of the original document. The original document is stored by GeoEngineers, Inc. and will serve as the official document of record.

Please refer to Appendix C, Report Limitations and Guidelines for Use for additional information pertaining to use of this report.

REFERENCES

American Association of State Highway and Transportation Officials, “LRFD Bridge Design Specifications.”

Booth, D.B., and Minard, J.P., “Geologic Map of the Issaquah 7.5 quadrangle, King County, Washington,” 1992.

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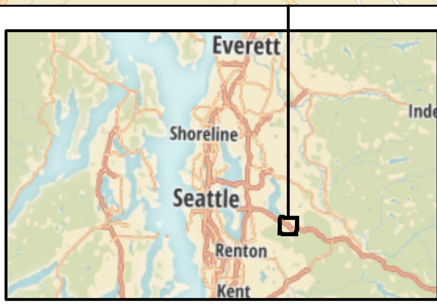
FIGURES



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Feet

Notes:
1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Mapbox Open Street Map, 2016

Projection: NAD 1983 UTM Zone 10N

Vicinity Map
Evergreen Ford Lincoln Issaquah, Washington
GeoEngineers — **Figure 1**

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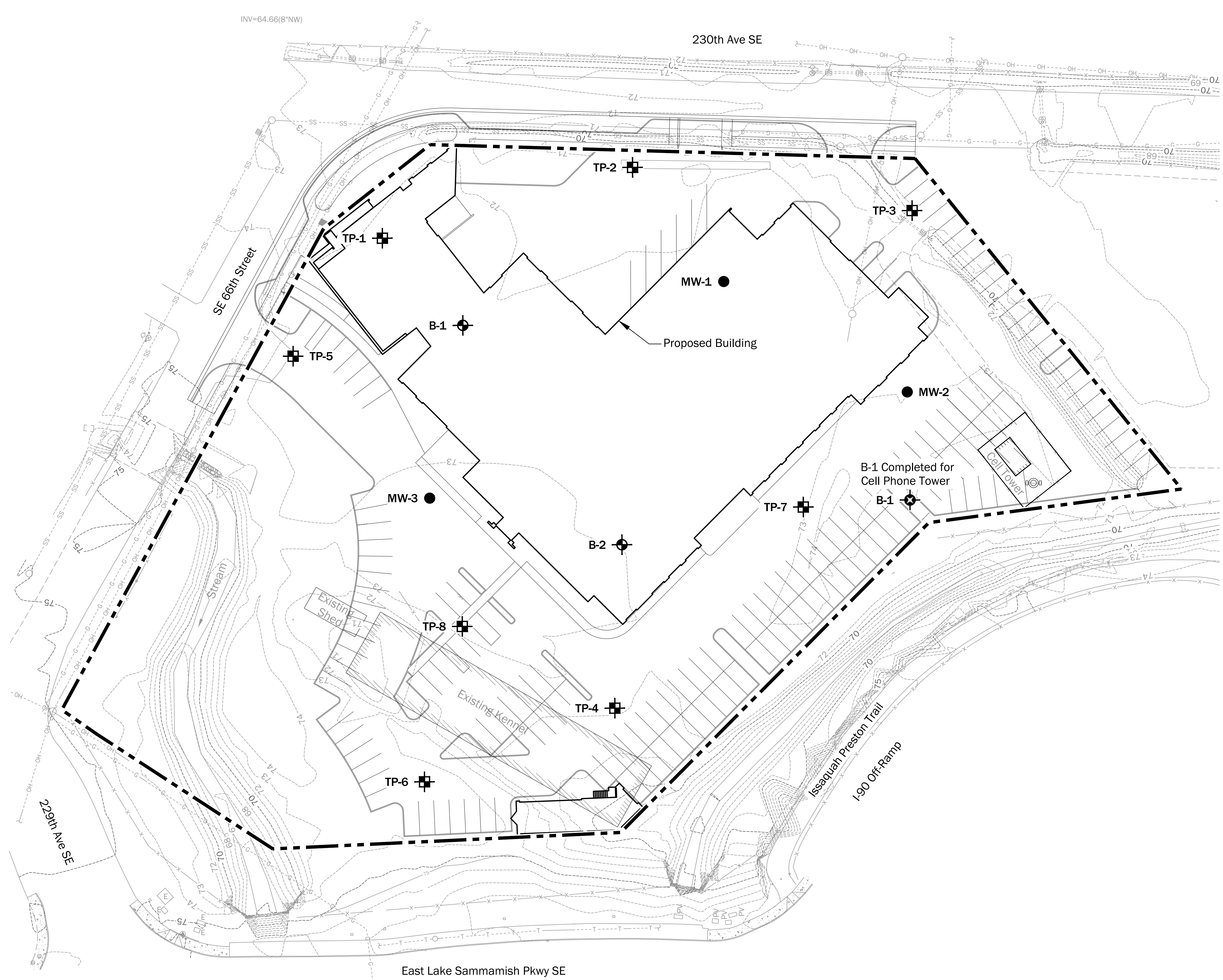

Legend

(dash-dot line)	Property Boundary
GEI-X	Boring by GeoEngineers, 2018
TP-X	Test Pit by GeoEngineers, 2018
MW-X	Monitoring Well by GeoEngineers, 2018
B-1	Boring by Others, 1997

Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Background received from SCJ Alliance on 12/3/2018.

Projection: WA State Plane, North Zone, NAD83, US Foot

N E S W

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Feet

Site Plan

Evergreen Ford Lincoln
Issaquah, Washington

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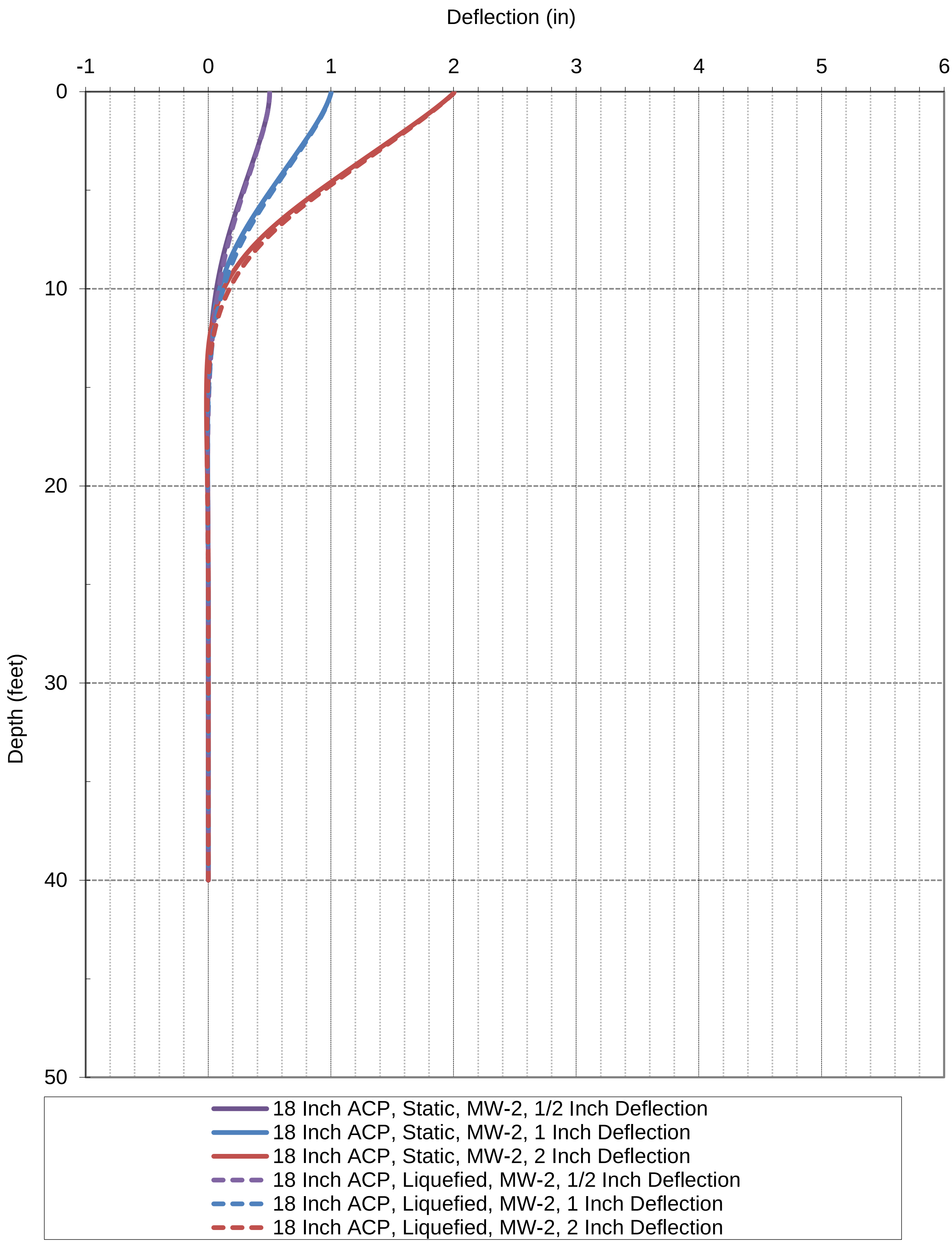
Figure 2



18 Inch Augercast Pile
Deflection vs. Depth MW-2

Evergreen Ford Lincoln
Issaquah, Washington

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Figure 3

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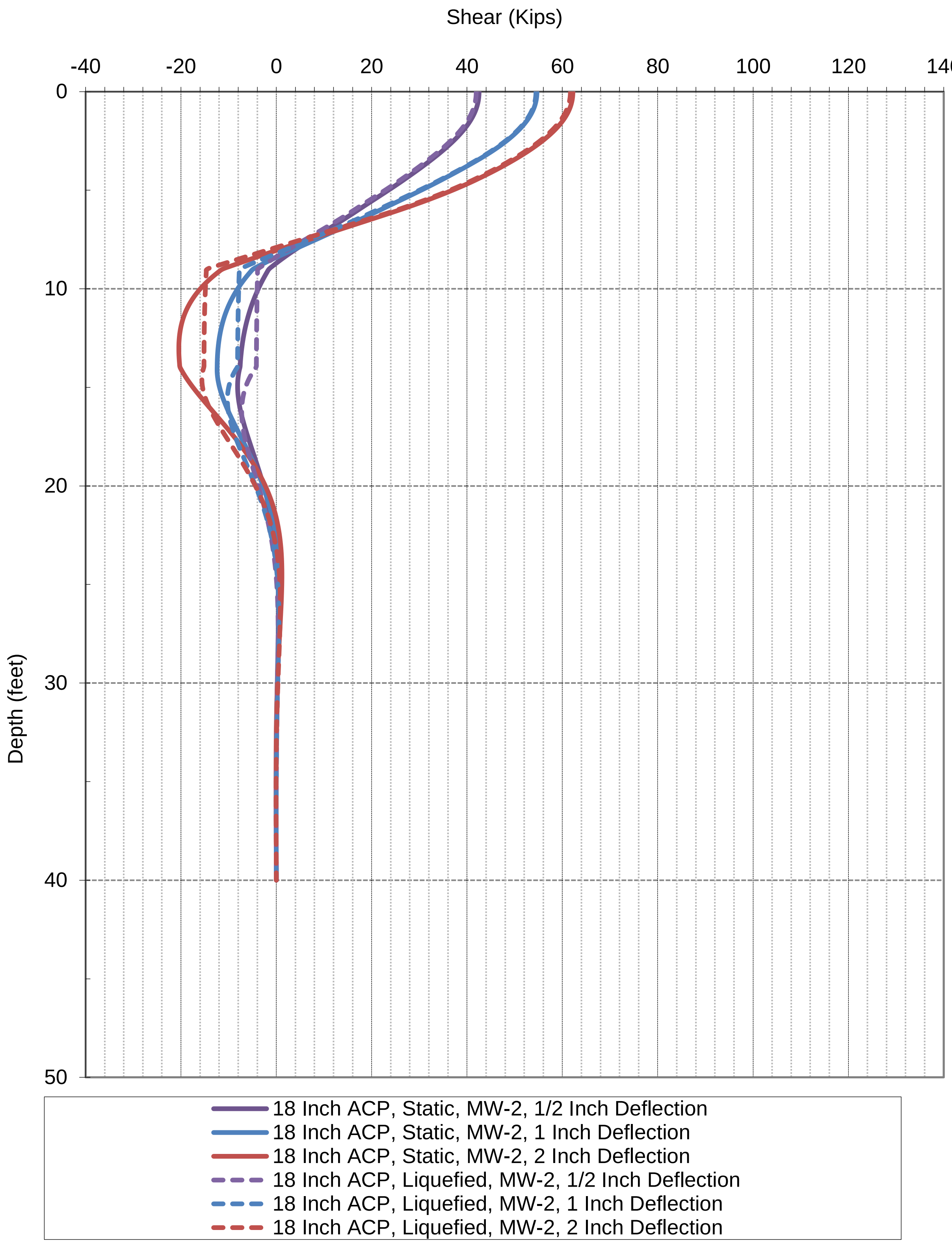

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18 Inch Augercast Pile
Shear vs. Depth MW-2

Evergreen Ford Lincoln
Issaquah, Washington

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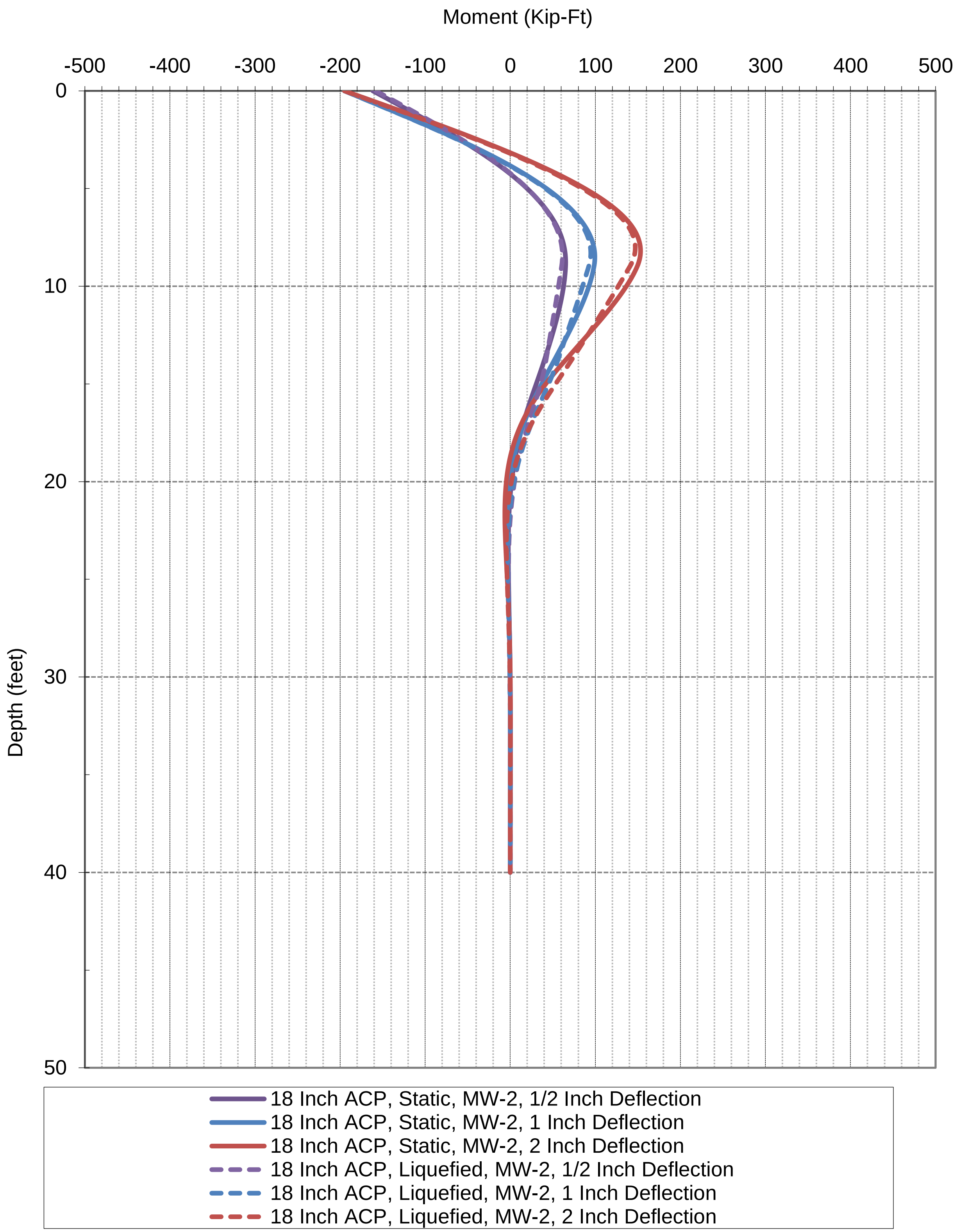
Figure 4



18 Inch Augercast Pile
Moment vs. Depth MW-2

Evergreen Ford Lincoln
Issaquah, Washington

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Figure 5

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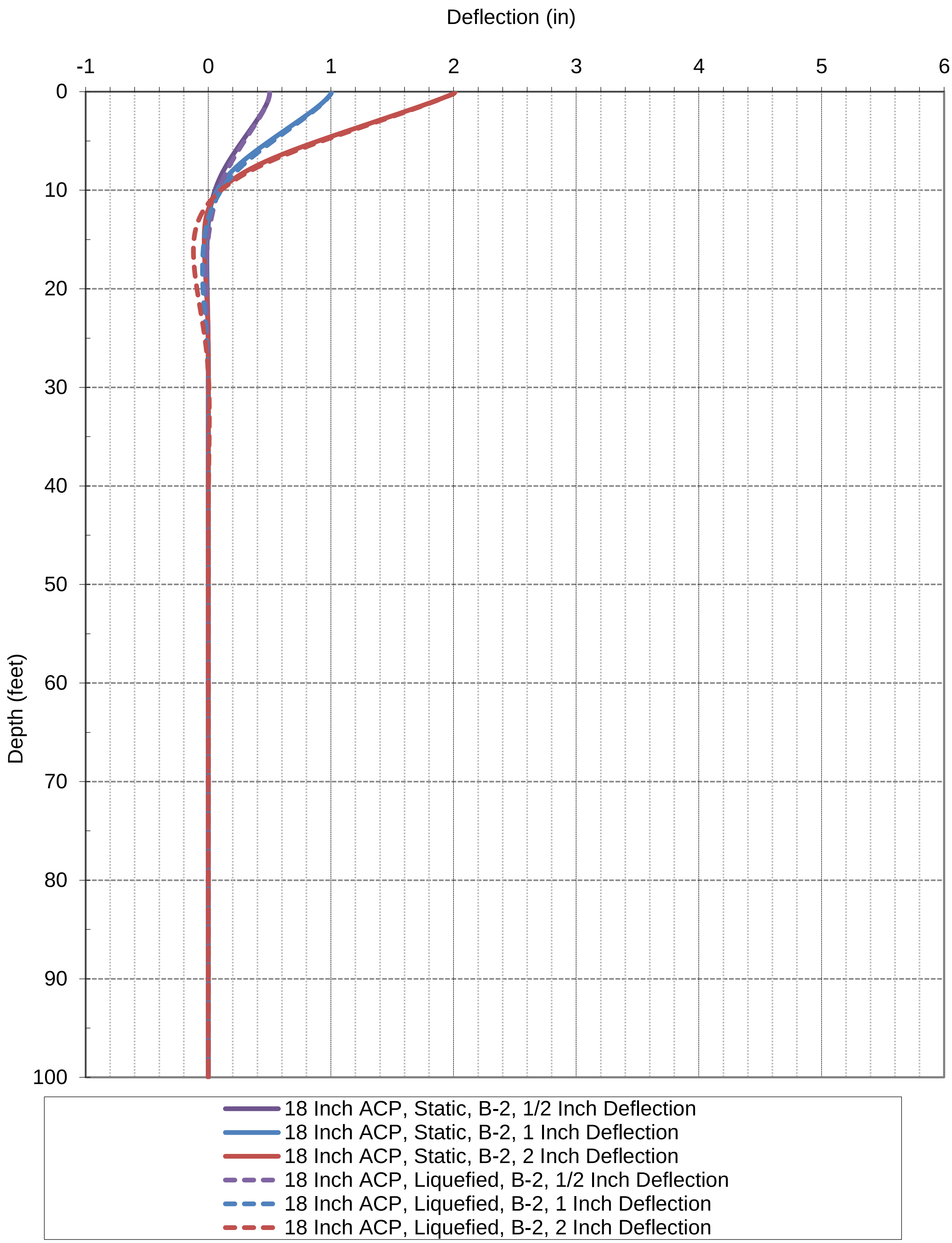

18 Inch Augercast Pile
Deflection vs. Depth B-2

Evergreen Ford Lincoln
Issaquah, Washington

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Figure 6

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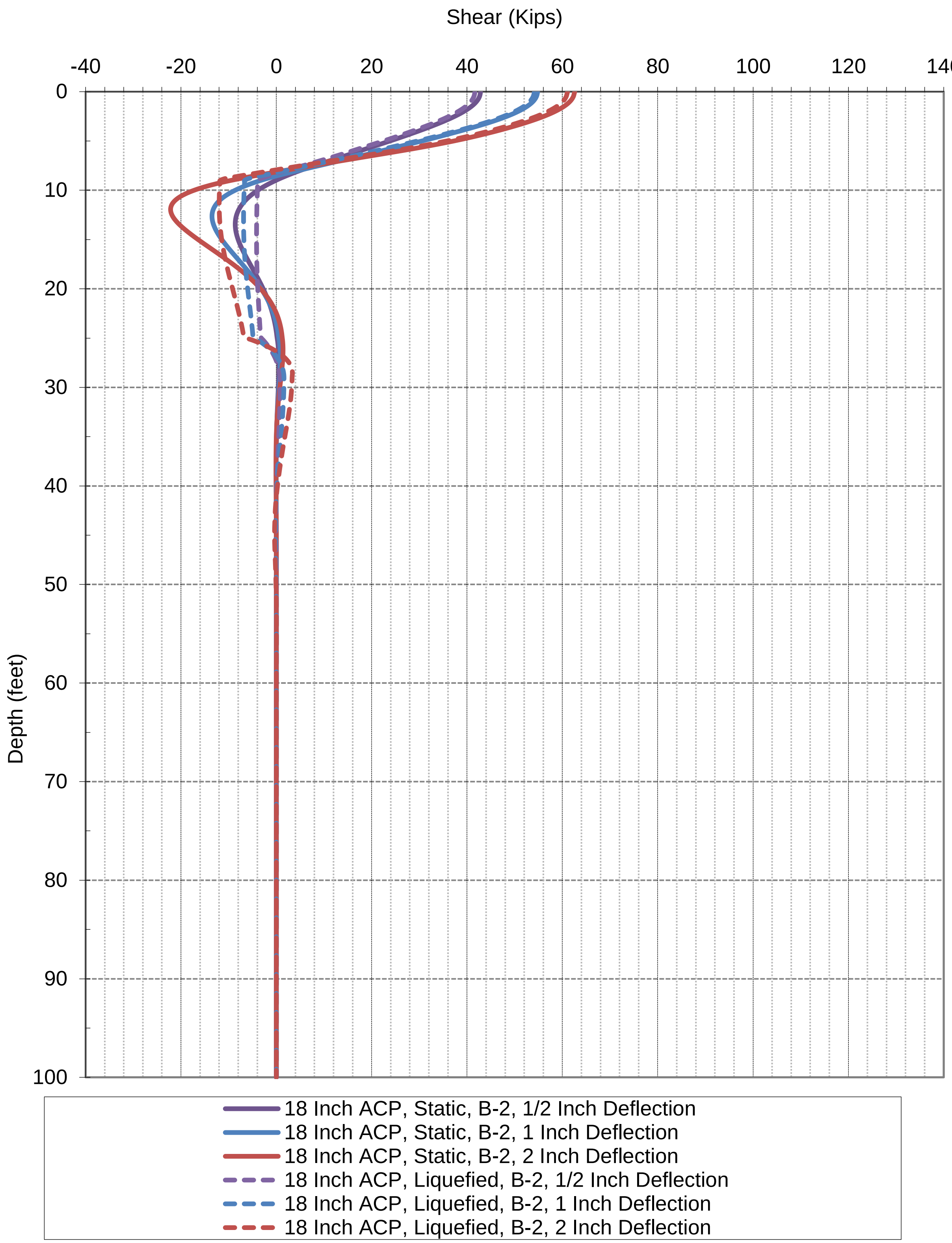

18 Inch Augercast Pile
Shear vs. Depth B-2

Evergreen Ford Lincoln
Issaquah, Washington

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Figure 7

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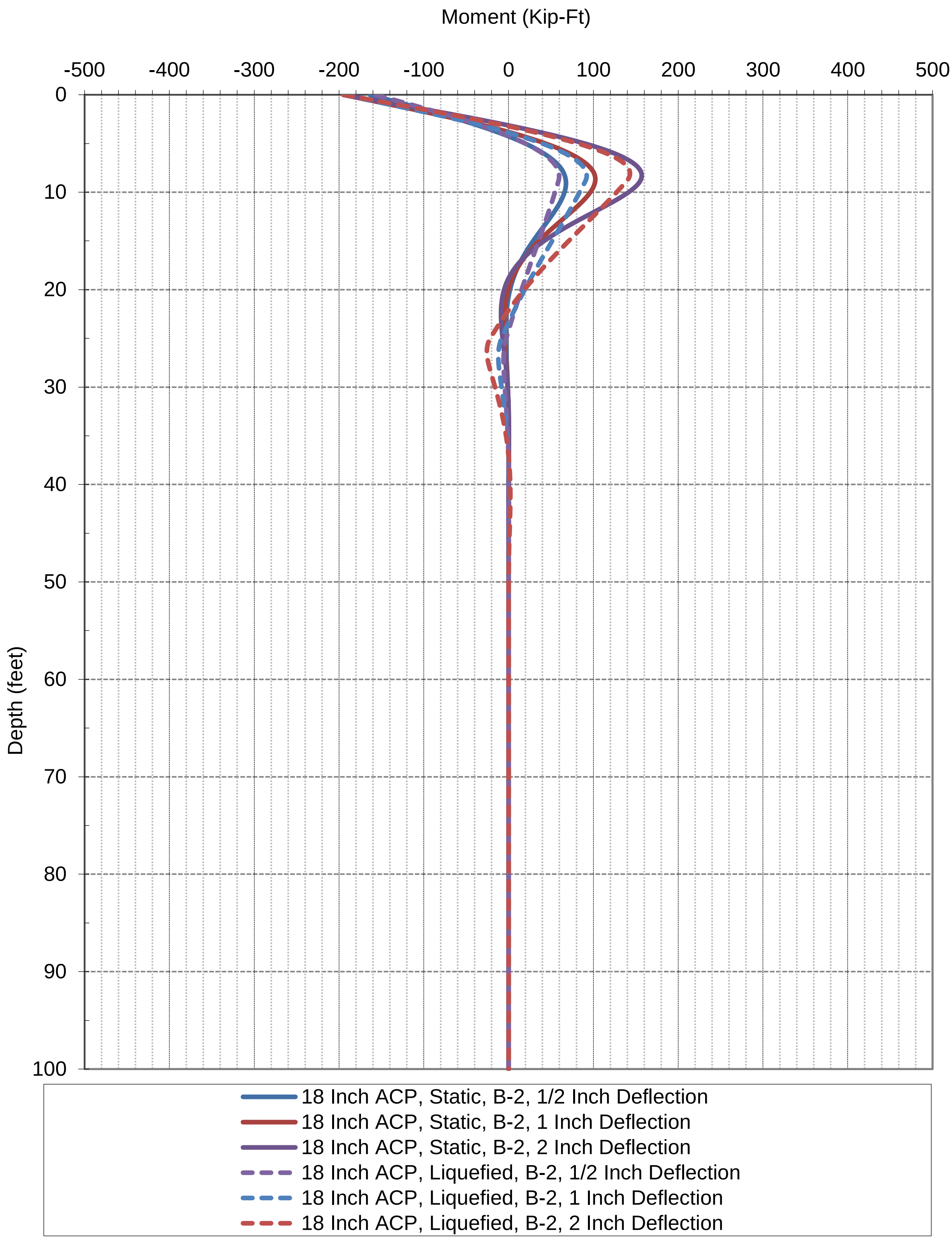

18 Inch Augercast Pile
Moment vs. Depth B-2

Evergreen Ford Lincoln
Issaquah, Washington

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Figure 8

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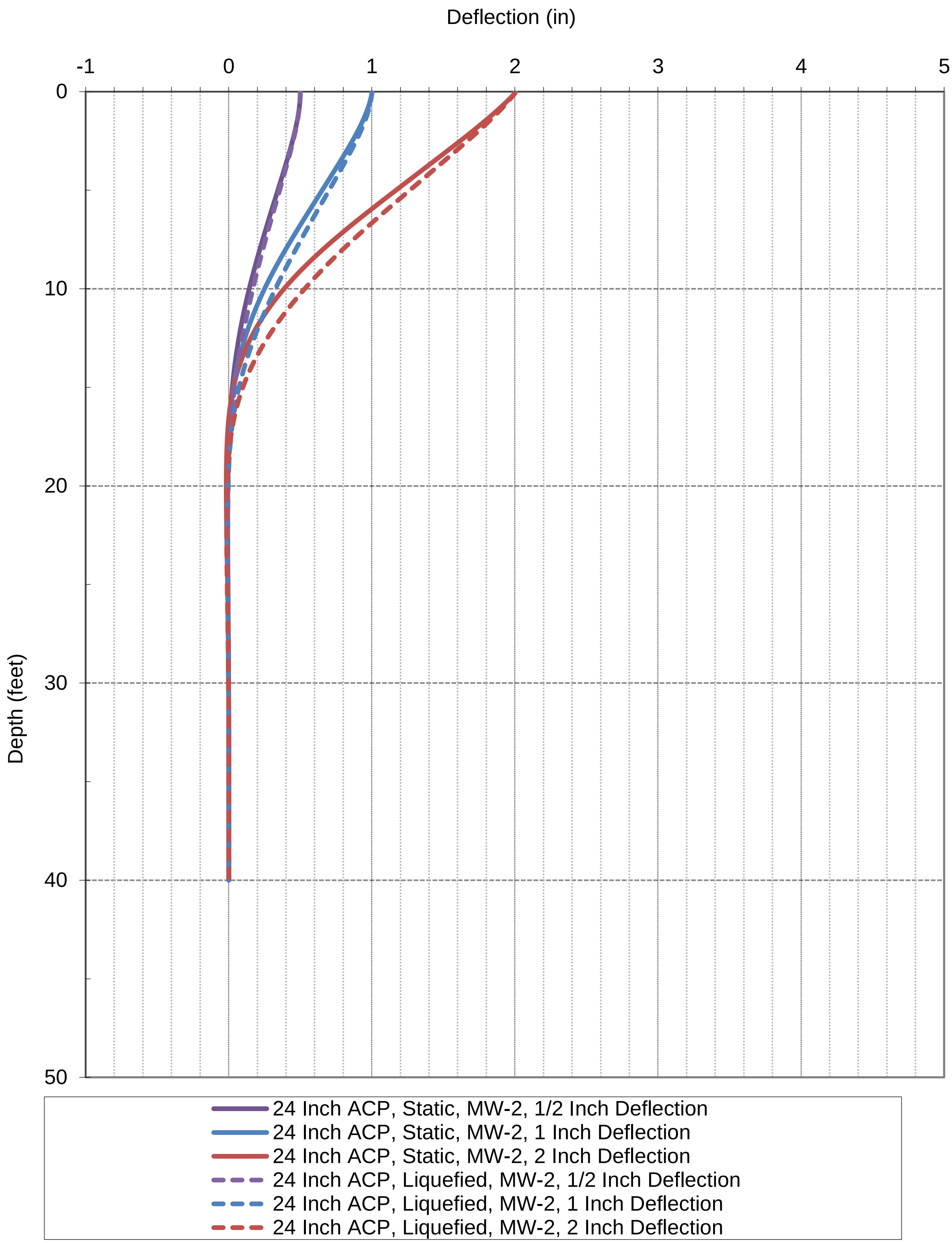

24 Inch Augercast Pile
Deflection vs. Depth MW-2

Evergreen Ford Lincoln
Issaquah, Washington

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Figure 9

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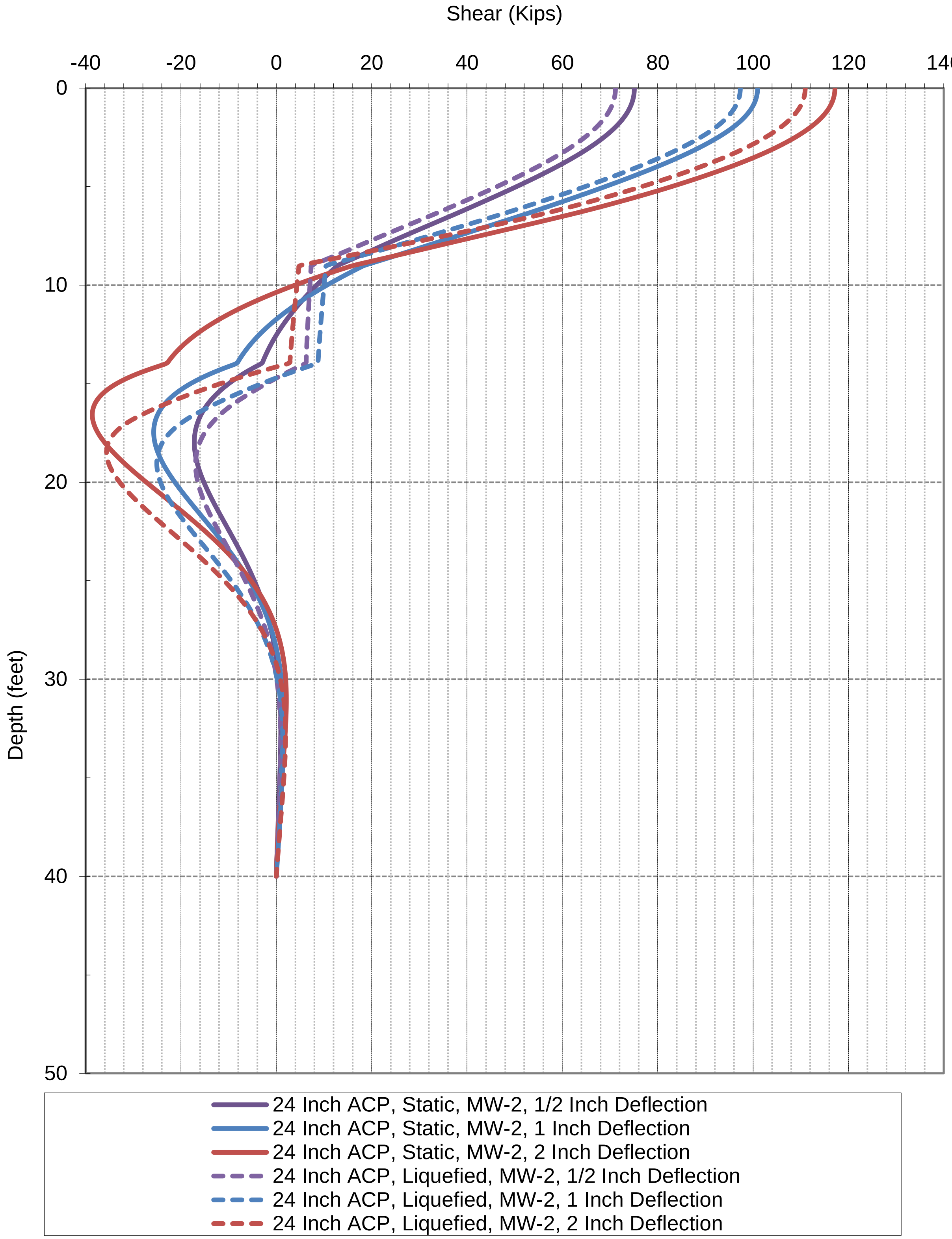

24 Inch Augercast Pile
Shear vs. Depth MW-2

Evergreen Ford Lincoln
Issaquah, Washington

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Figure 10

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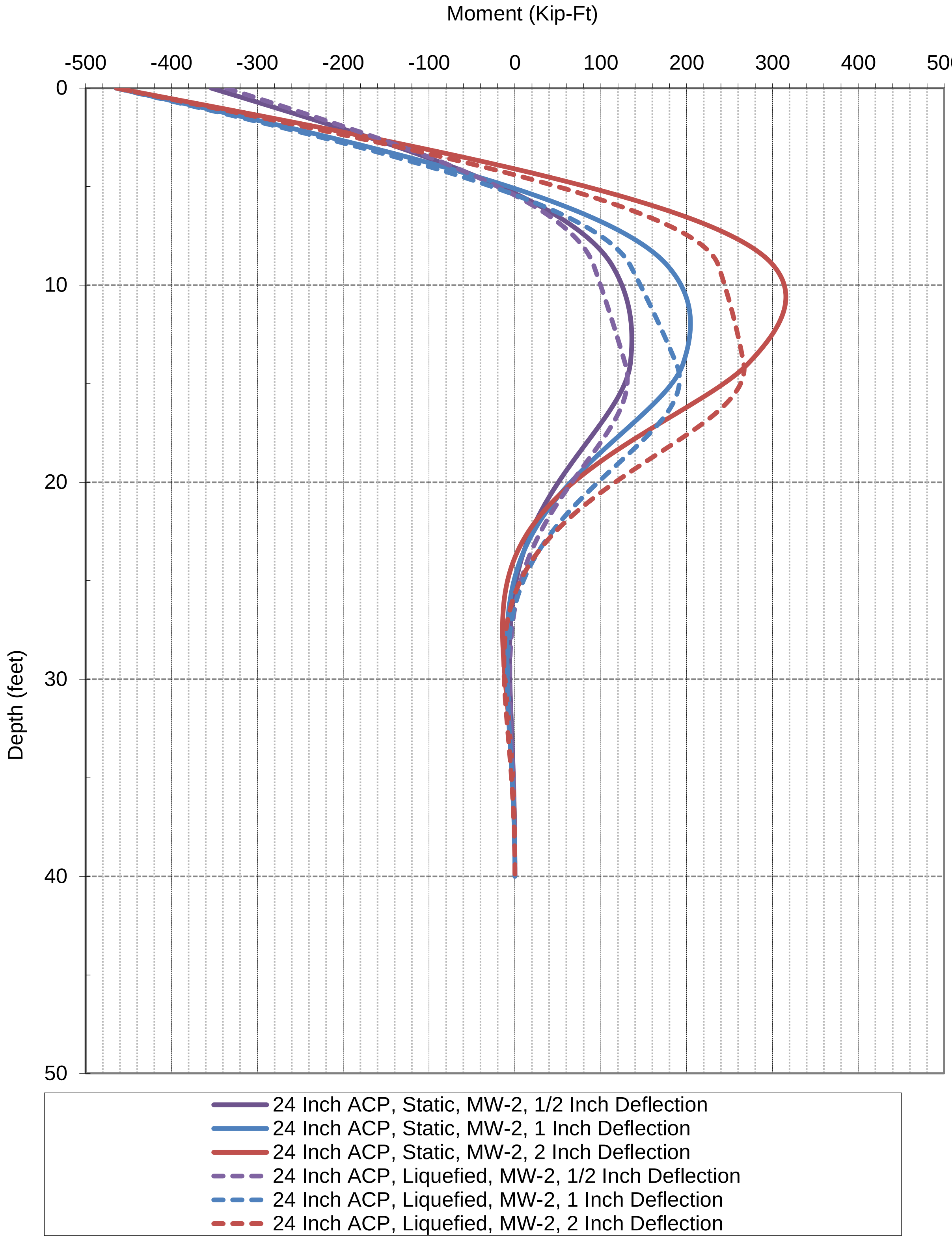

24 Inch Augercast Pile
Moment vs. Depth MW-2

Evergreen Ford Lincoln
Issaquah, Washington

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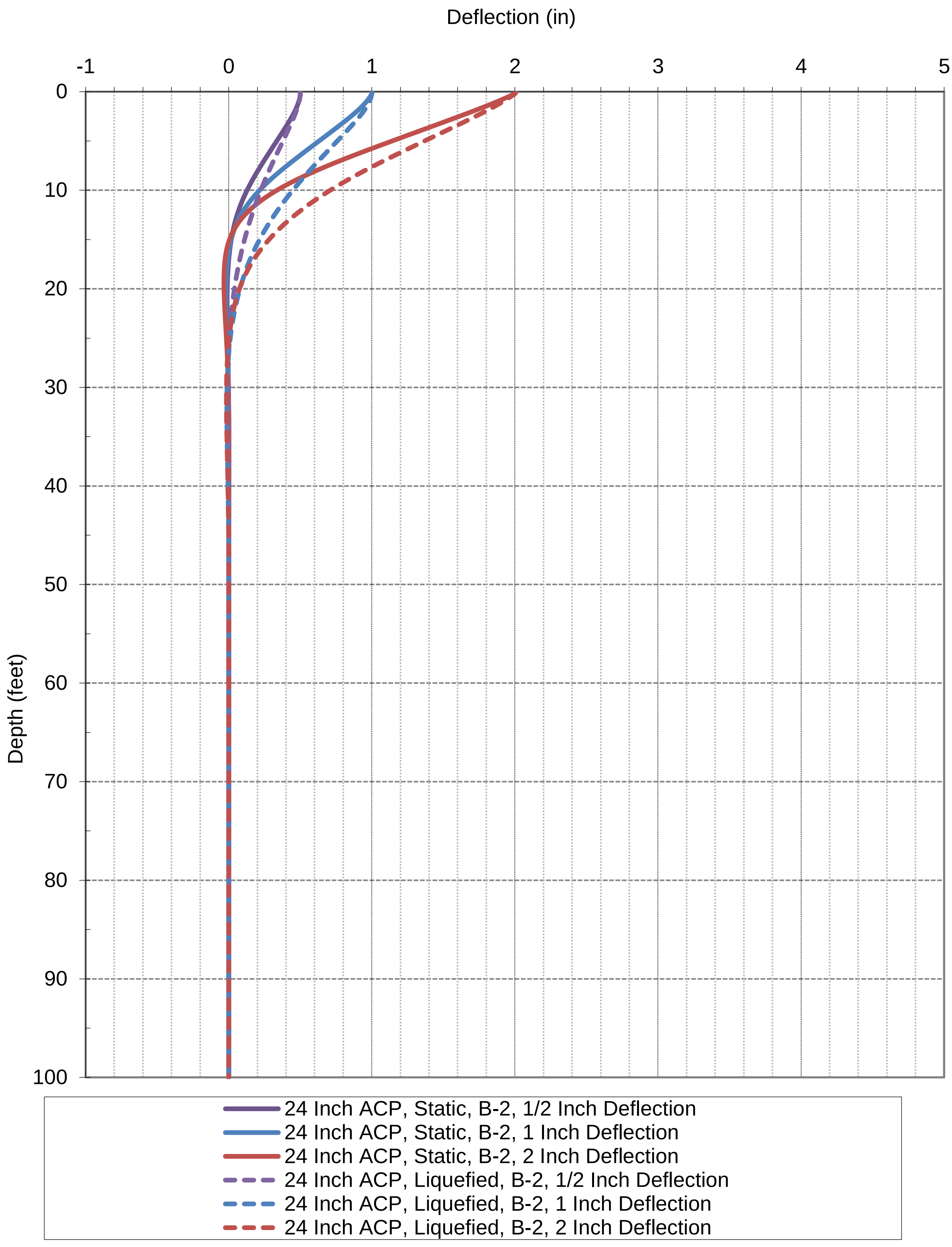
Figure 11

Sharepoint 23589-001-00\Technical Analysis DTM: 1/10/2019



24 Inch Augercast Pile
Deflection vs. Depth B-2

Evergreen Ford Lincoln
Issaquah, Washington

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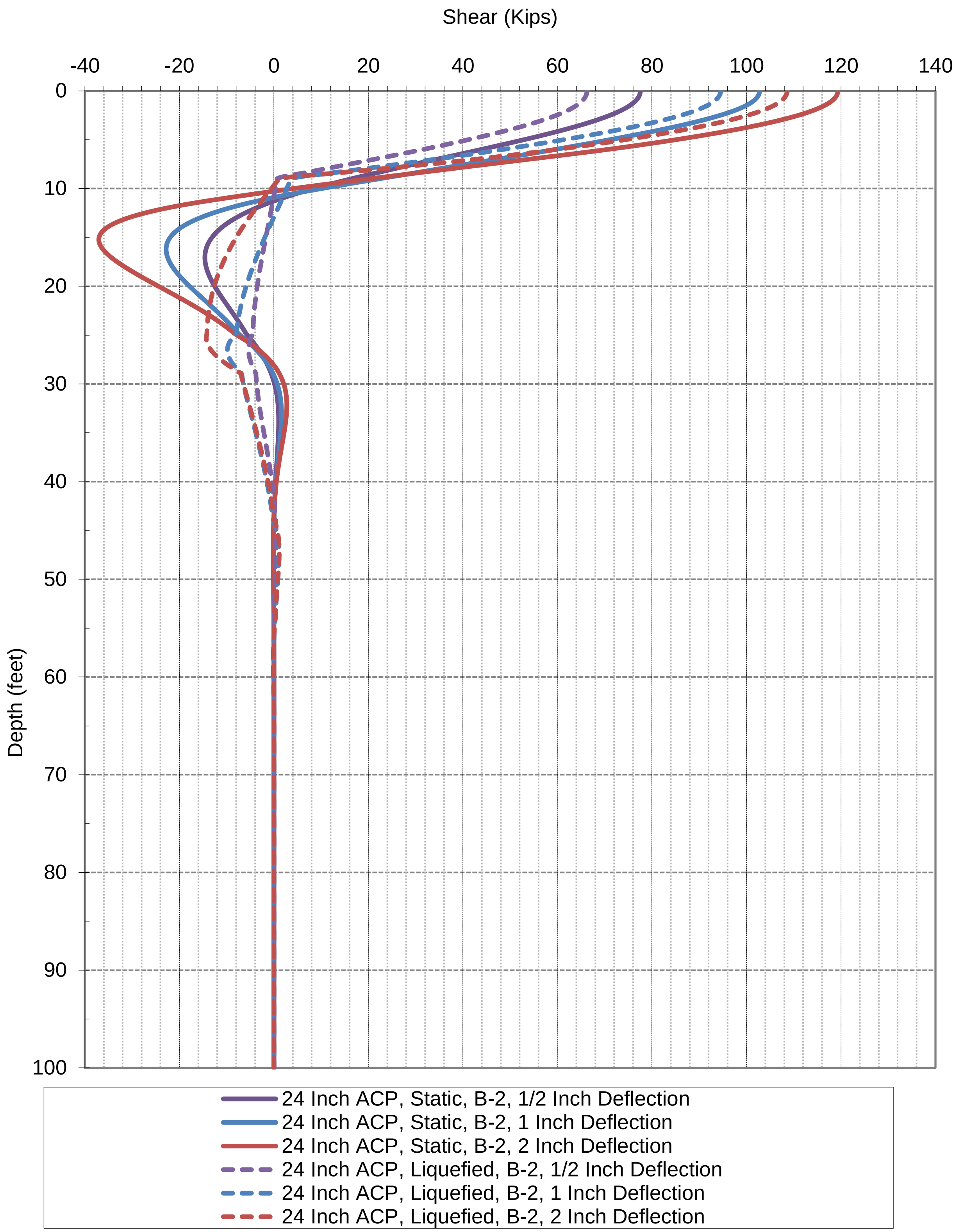
Figure 12

Sharepoint 23589-001-00\Technical Analysis DTM: 1/10/2019



24 Inch Augercast Pile
Shear vs. Depth B-2

Evergreen Ford Lincoln
Issaquah, Washington

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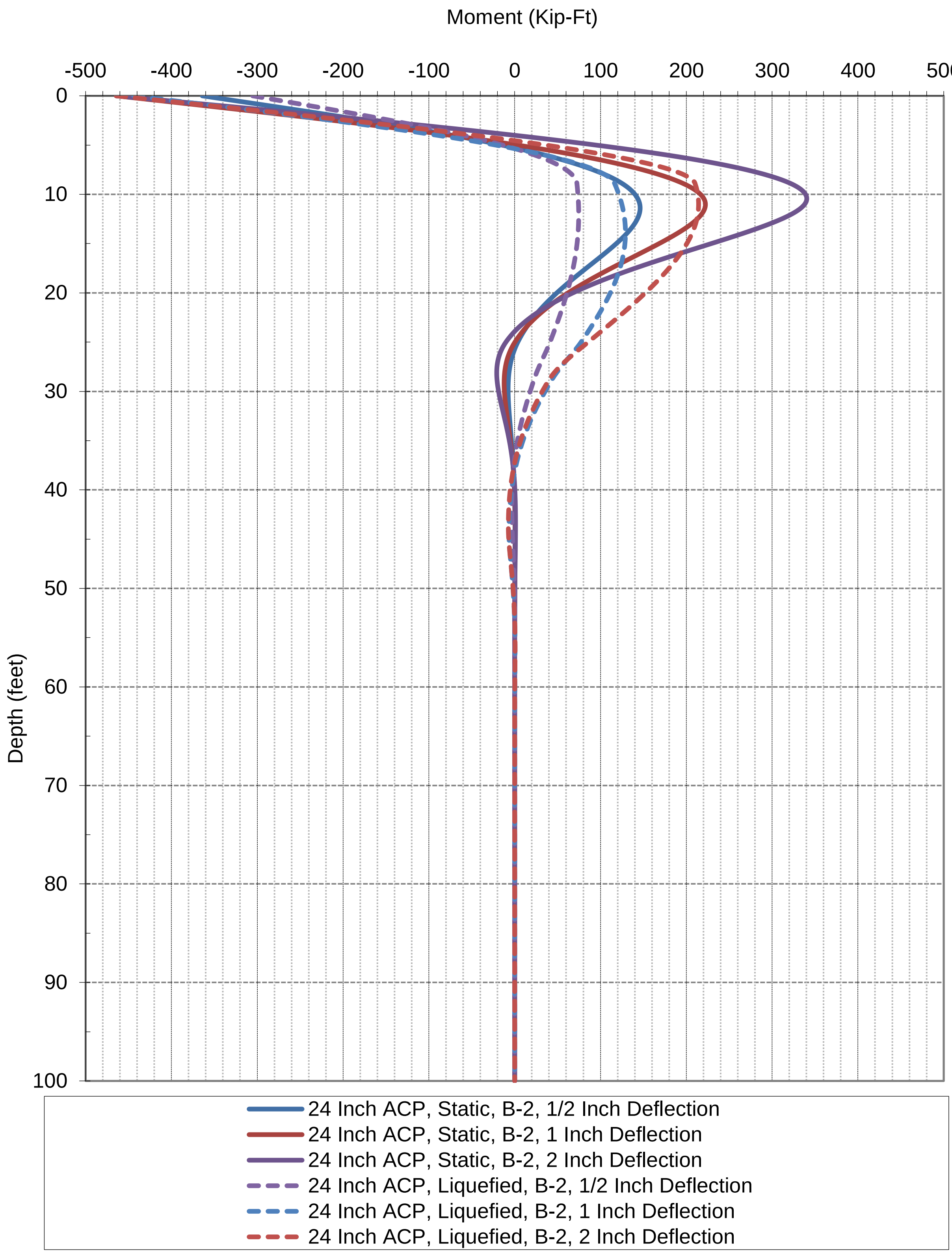
Figure 13

Sharepoint 23589-001-00\Technical Analysis DTM: 1/10/2019



24 Inch Augercast Pile
Moment vs. Depth B-2

Evergreen Ford Lincoln
Issaquah, Washington

GEOENGINEERS

Figure 14

Sharepoint 23589-001-00\Technical Analysis DTM: 1/10/2019

APPENDICES

APPENDIX A
Field Explorations and Laboratory Testing

APPENDIX A
FIELD EXPLORATIONS AND LABORATORY TESTING

Field Explorations

Subsurface conditions at the site were explored on October 31 through November 2, 2018 by drilling five borings/monitoring wells (MW-1 through MW-3 and B-1 and B-2) at the approximate locations shown on Figure 2, and by completing eight test pits across the site. The approximate exploration locations were established in the field by measuring distances from existing site features and using a handheld GPS. The explorations were completed to depths between 5 and 50 feet using track-mounted equipment owned and operated by Saber and Advanced Drill Technologies.

Borings/Monitoring Wells

Disturbed soils samples were obtained during drilling using standard penetration test (SPT) methodology with the standard split-spoon sampler in the borings. The samples were placed in plastic bags to maintain the moisture content and transported back to our laboratory for analysis and testing.

The borings were continuously monitored by a geologist from our firm who examined and classified the soils encountered, obtained representative soil samples, observed groundwater conditions and prepared a detailed log of each exploration. Soils encountered were classified visually in general accordance with ASTM D2488-09a the classification system described in Figure A-1. An explanation of our boring log symbols is also shown on Figure A-1.

The logs of the borings are presented in Figures A-2 through A-6. The exploration logs are based on our interpretation of the field and laboratory data and indicate the various types of soils encountered. The logs also indicate the depths at which these soils or their characteristics change, although the change might actually be gradual. If the change occurred between samples in the boring, it was interpreted.

Test Pits

Eight test pit explorations were completed to observe shallow surface conditions such as thickness of fill, groundwater seepage, soil density, and existence of compressible soils. Soil description, probe depths, groundwater observations, caving conditions, and field measured shear strength measurements are recorded on test pit logs. The logs of the test pits are presented in Figures A-7 through A-14.

Laboratory Testing

Soil samples obtained from the explorations were transported to our laboratory and examined to confirm or modify field classifications, as well as to evaluate index properties of the soil samples. Representative samples were selected for laboratory testing consisting of the determination of the moisture content, sieve analyses, and percent fines. The tests were performed in general accordance with test methods of the American Society for Testing and Materials (ASTM) or other applicable procedures.

Moisture Content Testing

Moisture content tests were completed in general accordance with ASTM D 2216 for representative samples obtained from the explorations. The results of these tests are presented on the exploration logs at the depths at which the samples were obtained.

Sieve Analyses

Sieve analyses were performed on selected samples in general accordance with ASTM D 422 to determine the sample grain size distribution. The wet sieve analysis method was used to determine the percentage of soil greater than the U.S. No. 200 mesh sieve. The results of the sieve analyses were plotted, classified in general accordance with the Unified Soil Classification System (USCS), and are presented in Figures A-15 and A-16.

Percent Fines Test

Percent fines (particles passing the No. 200 sieve) were completed on soil samples using ASTM D 1140. The wet sieve method was used to determine the percentage of soil particles larger than the U.S. No. 200 sieve opening. The results of the percent fines tests are presented on the boring logs at the depths at which the samples were obtained.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% RETAINED ON NO. 200 SIEVE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS
				SP	POORLY-GRADED SANDS, GRAVELLY SAND
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% PASSING NO. 200 SIEVE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS, ROCK FLOUR, CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS AND SILTS OF MEDIUM TO HIGH PLASTICITY
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: Multiple symbols are used to indicate borderline or dual soil classifications

ADDITIONAL MATERIAL SYMBOLS

SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
	AC	Asphalt Concrete
	CC	Cement Concrete
	CR	Crushed Rock/ Quarry Spalls
	SOD	Sod/Forest Duff
	TS	Topsoil

Groundwater Contact

Measured groundwater level in exploration, well, or piezometer

Measured free product in well or piezometer

Graphic Log Contact

Distinct contact between soil strata

Approximate contact between soil strata

Material Description Contact

Contact between geologic units

Contact between soil of the same geologic unit

Sampler Symbol Descriptions

2.4-inch I.D. split barrel

Standard Penetration Test (SPT)

Shelby tube

Piston

Direct-Push

Bulk or grab

Continuous Coring

Blowcount is recorded for driven samplers as the number of blows required to advance sampler 12 inches (or distance noted). See exploration log for hammer weight and drop.

"P" indicates sampler pushed using the weight of the drill rig.

"WOH" indicates sampler pushed using the weight of the hammer.

Laboratory / Field Tests

%F	Percent fines
%G	Percent gravel
AL	Atterberg limits
CA	Chemical analysis
CP	Laboratory compaction test
CS	Consolidation test
DD	Dry density
DS	Direct shear
HA	Hydrometer analysis
MC	Moisture content
MD	Moisture density
Mohs	Mohs hardness scale
OC	Organic content
PM	Permeability or hydraulic conductivity
PI	Plasticity index
PP	Pocket penetrometer
SA	Sieve analysis
TX	Triaxial compression
UC	Unconfined compression
VS	Vane shear

Sheen Classification

NS	No Visible Sheen
SS	Slight Sheen
MS	Moderate Sheen
HS	Heavy Sheen

NOTE: The reader must refer to the discussion in the report text and the logs of explorations for a proper understanding of subsurface conditions. Descriptions on the logs apply only at the specific exploration locations and at the time the explorations were made; they are not warranted to be representative of subsurface conditions at other locations or times.

Key to Exploration Logs

GEOENGINEERS

Figure A-1

Drilled	Start 11/1/2018	End 11/1/2018	Total Depth (ft)	51.5	Logged By WCW Checked By MSH	Driller Advanced Drill Technologies	Drilling Method	Hollow-stem Auger

Surface Elevation (ft) Vertical Datum	72 NAVD88	Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	Diedrich D50 Turbo
Latitude Longitude	47.54232 -122.03412	System Datum	WA State Plane North NAD83 (feet)	See "Remarks" section for groundwater observed	

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
72	0							SM	Brown silty fine sand, oxidation staining (loose, moist) (fill)			
70			12	5		1						
	5		10	7		2a 2b MC			Becomes tan	28		
65								GM	Brown silty fine to coarse gravel with sand (medium dense, wet)			Groundwater observed at approximately 8½ feet at time of drilling
	10		10	13		3a 3b						
60									Grades without gravel			
								SP-SM	Gray fine to coarse sand with silt and gravel (medium dense, wet)			
	15		8	13		4 %F				12	5	
55								SM	Gray-blue silty fine sand with occasional gravel (medium dense, wet)			
	20		18	13		5a 5b						Driller added mud
50								SM	Blue-gray silty fine to coarse sand with gravel (medium dense to dense, wet)			
	25		4	30		6						
45								SM	Brown silty fine sand with 2-inch sand lens (loose, wet)			
	30		18	8		7						
40								SM	Brown silty fine to coarse sand with occasional gravel (medium dense, wet)			
	35											

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Topographic Survey. Vertical approximated based on Topographic Survey.

Log of Boring B-1

GEOENGINEERS

Project: Evergreen Ford Lincoln
Project Location: 22909 SE 66th Street, Issaquah, Washington
Project Number: 23589-001-00

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	35		12	14		8						Silt content decreased with depth as sand content increased, gravel at top of sample
35								GP-GM	Brown fine to coarse gravel with silt, sand and occasional cobbles (dense to very dense, wet)			
	40		16	49		9						
30												
	45		10	84		10						
25								SM	Brown silty fine to medium sand interlayered with silty gravel and coarse sand (dense, wet)			
	50		12	34		11						

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Log of Boring B-1 (continued)

GeoEngineers

Project: Evergreen Ford Lincoln

Project Location: 22909 SE 66th Street, Issaquah, Washington

Project Number: 23589-001-00

Figure A-2

Sheet 2 of 2

Drilled	Start 11/1/2018	End 11/1/2018	Total Depth (ft)	81.5	Logged By WCW Checked By MSH	Driller	Advanced Drill Technologies	Drilling Method	Hollow-stem Auger

Surface Elevation (ft) Vertical Datum	73 NAVD88	Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	Diedrich D50 Turbo
Latitude Longitude	47.54206 -122.03464	System Datum	WA State Plane North NAD83 (feet)	See "Remarks" section for groundwater observed	

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Recovered (in)	Blows/foot	Sample Name / Testing	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0				SM	Brown-gray silty fine to coarse sand with gravel (loose, moist) (fill)			
70		12	8	1 MC			9		
	5	10	6	2a 2b		Becomes dark brown			
65					GP-GM	Brown fine to coarse gravel with silt and sand (medium dense, wet)			Groundwater observed at approximately 9 feet at time of drilling
	10	12	15	3					
60					SM	Dark gray silty fine to coarse sand with gravel (medium dense, wet)			
	15	6	16	4					
55					OL	Dark brown organic silt with sand (soft, wet)			
	20	18	3	5a MC 5b MC	SM	Silty fine to medium sand with occasional gravel (very loose, wet)	56 23		Driller added mud
50									
	25	13	33	6a 6b	GP-GM	Brown fine to coarse gravel with silt and sand (dense, wet)			
45					SM	Brown silty fine to coarse sand with gravel (loose to medium dense, wet)			
	30	18	9	7 %F			17	15	
40									
	35								

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Topographic Survey. Vertical approximated based on Topographic Survey.

Log of Boring B-2

GEOENGINEERS

Project: Evergreen Ford Lincoln
Project Location: 22909 SE 66th Street, Issaquah, Washington
Project Number: 23589-001-00

Figure A-3
Sheet 1 of 3

Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
35	X	7	16		8					
						GM	Brown silty gravel with sand (medium dense, wet)			
40	X	12	14		9a 9b					
						SM	Brown silty fine to coarse sand with gravel (medium dense, wet)			
						GP-GM	Brown fine to coarse gravel with silt and sand (dense, wet)			
45	X	12	35		10					
50	X	12	48		11a 11b					
						SP-SM	Brown fine to coarse sand with silt and gravel (medium dense, wet)			
55	X	12	28		12 / %F			10	7	
60	X	12	10		13					
						SM/OL	Gray silty fine sand interlayered with gray organic silt and trace wood debris (medium dense, moist)			
65	X	18	13		14 / MC			37		
						SM	Dark gray silty fine sand (medium dense, wet)			
70	X	18	23		15 / MC			27		
75	X	18	22		16		Oxidation staining			

Elevation (feet): 35, 30, 25, 20, 15, 10, 5, 0

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Log of Boring B-2 (continued)

GeoEngineers

Project: Evergreen Ford Lincoln
Project Location: 22909 SE 66th Street, Issaquah, Washington
Project Number: 23589-001-00

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
-5								SM	Blue-gray silty fine sand (dense, moist)			
	80		18	39		17a 17b						
								GM	Brown-gray silty fine to medium gravel with sand (dense, moist)			

Log of Boring B-2 (continued)

GeoEngineers

Project: Evergreen Ford Lincoln
Project Location: 22909 SE 66th Street, Issaquah, Washington
Project Number: 23589-001-00

Figure A-3
Sheet 3 of 3

Drilled	Start	End	Total Depth (ft)	Logged By	Checked By	Driller	Drilling Method
	11/2/2018	11/2/2018	31.5	WCW	MSH	Advanced Drill Technologies	Hollow-stem Auger

Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	Diedrich D50 Turbo
Surface Elevation (ft) / Vertical Datum	72 / NAVD88	Top of Casing Elevation (ft)	
Latitude / Longitude	47.5419 / -122.03401	Horizontal Datum	WA State Plane North NAD83 (feet)

A 2-in well was installed on 11/2/2018 to a depth of 14 ft.

Groundwater Date Measured	Depth to Water (ft)	Elevation (ft)
1/15/2019	6.50	65.50

Notes:

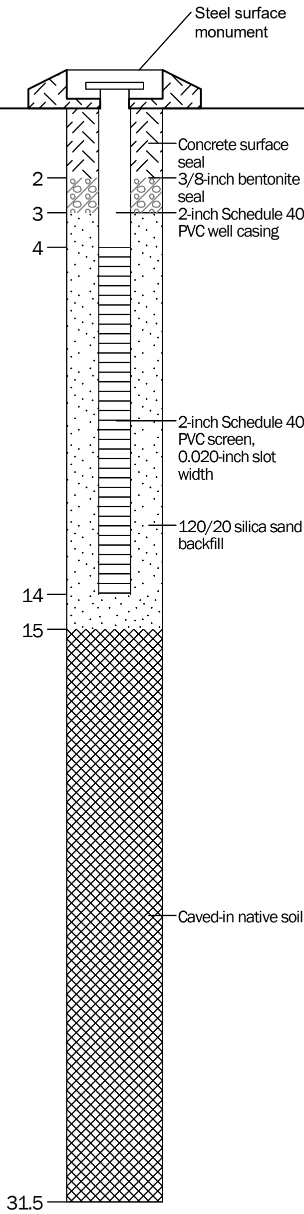
FIELD DATA

Depth (feet)	Recovered (in)	Blows/foot	Sample Name / Testing	Moisture Content (%)
2.5	18	14	1 / MC	17
5	10	16	2	
10	8	17	3	
15	8	9	4	
20	10	33	5	
25	8	50/5"	6	
30	10	57	7a, 7b	

MATERIAL DESCRIPTION

Group Classification	Material Description
SM	Brown-tan silty fine to coarse sand (medium dense, moist) (fill)
	Grades with gravel
SP-SM	Blue-gray fine to coarse sand with silt and gravel (medium dense, moist)
SM	Blue-gray silty fine to coarse sand with gravel (loose, wet)
GP-GM	Blue-gray fine to coarse gravel with silt and sand (dense, wet)
SP-SM	Bluish gray fine to coarse sand with silt and gravel (very dense, wet)
SP	Gray fine to medium sand (very dense, wet)
GP-GM	Tan-brown fine to coarse gravel with silt and sand (very dense, wet)

WELL LOG



Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Topographic Survey. Vertical approximated based on Topographic Survey.

Log of Monitoring Well MW-1

GeoEngineers

Project: Evergreen Ford Lincoln
Project Location: 22909 SE 66th Street, Issaquah, Washington
Project Number: 23589-001-00

Figure A-4
Sheet 1 of 1

Drilled	Start	End	Total Depth (ft)	Logged By	Checked By	Driller	Drilling Method
	11/2/2018	11/2/2018	31.5	WCW	MSH	Advanced Drill Technologies	Hollow-stem Auger

Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	Diedrich D50 Turbo
Surface Elevation (ft) Vertical Datum	73 NAVD88	Top of Casing Elevation (ft)	
Latitude Longitude	47.546 -122.03427	Horizontal Datum	WA State Plane North NAD83 (feet)

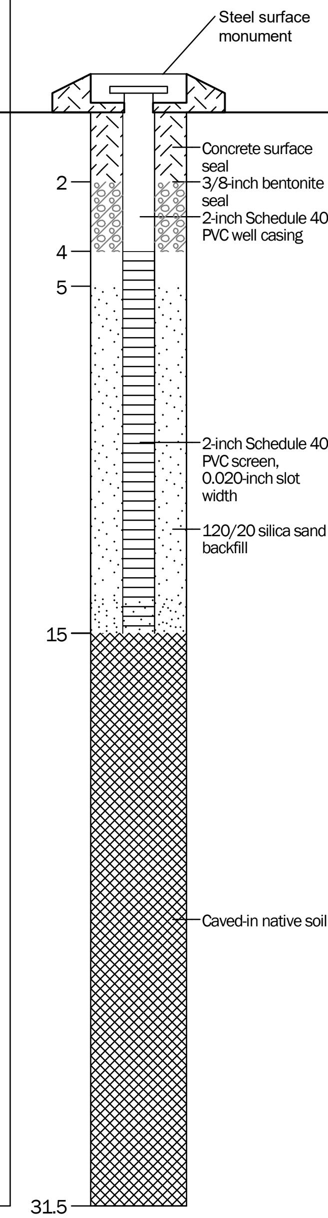
A 2-in well was installed on 11/2/2018 to a depth of 15 ft.

Groundwater Date Measured	Depth to Water (ft)	Elevation (ft)
1/15/2019	7.85	65.15

Notes:

Elevation (feet)	Depth (feet)	Recovered (in)	Blows/foot	Sample Name Testing	Group Classification	Material Description	Moisture Content (%)	Fines Content (%)
	0				SM	Brown silty fine to coarse sand with gravel and trace organic matter (medium dense, moist) (fill)		
70		18	18	1 %F			17	40
					SP-SM	Brown fine to coarse sand with silt, occasional gravel and wood debris (loose, moist)		
	5	18	5	2 SA			15	6
65								
					ML	Blue-gray sandy silt, oxidation staining (soft, moist)		
	10	14	3	3a 3b %F				
					SM	Blue-gray silty fine to medium sand with pockets of silt, oxidation staining (very loose, moist)	34	30
60								
	15	16	29	4		Becomes medium dense and with gravel		
55								
					SP-SM	Brown-gray fine to coarse sand with silt and occasional gravel (dense, wet)		
	20	16	33	5				
50								
	25	10	50/4"	6		Becomes very dense		
45								
					SP	Brown-gray fine to medium sand (very dense, wet)		
	30	14	50	7				

WELL LOG



Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Topographic Survey. Vertical approximated based on Topographic Survey.

Log of Monitoring Well MW-2

GeoEngineers

Project: Evergreen Ford Lincoln
Project Location: 22909 SE 66th Street, Issaquah, Washington
Project Number: 23589-001-00

Drilled	Start 11/2/2018	End 11/2/2018	Total Depth (ft)	46.5	Logged By WCW / Checked By MSH	Driller	Advanced Drill Technologies	Drilling Method	Hollow-stem Auger

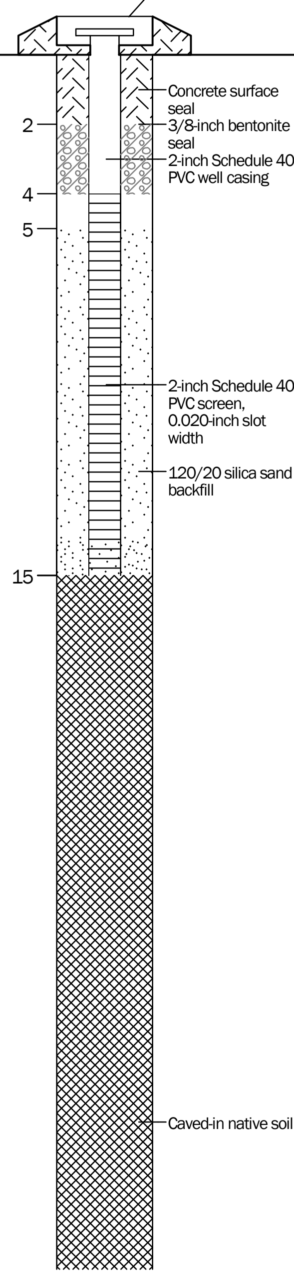
Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	Diedrich D50 Turbo	A 2-in well was installed on 11/2/2018 to a depth of 15 ft.
Surface Elevation (ft) / Vertical Datum	73 / NAVD88	Top of Casing Elevation (ft)		
Latitude / Longitude	47.54237 / -122.03454	Horizontal Datum	WA State Plane North NAD83 (feet)	

Groundwater Date Measured	Depth to Water (ft)	Elevation (ft)
1/15/2019	7.05	65.95

Notes:

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Sample Name / Testing	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)
73	0					SM	Brown to blue silty fine to coarse sand with gravel, and trace organic matter (medium dense, moist) (fill)		
70		X	16	18	1				
	5	X	16	8	2 / SA	SM	Brown silty fine sand with gravel, oxidation staining and organic matter (stiff, moist)	34	30
65									
	10	X	0	2	NR				
60						SP-SM	Blue gray fine to coarse sand with silt, gravel and occasional cobbles (loose to medium dense, wet)		
	15	X	6	19	3				
55									
	20	X	12	8	4 / %F			18	11
50									
	25	X	12	33	5				
45									
	30	X	14	14	6				
40									
	35								

WELL LOG



Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Topographic Survey. Vertical approximated based on Topographic Survey.

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Log of Monitoring Well MW-3

GeoEngineers

Project: Evergreen Ford Lincoln
Project Location: 22909 SE 66th Street, Issaquah, Washington
Project Number: 23589-001-00

Figure A-6

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	WELL LOG
	35		3	12		7				Grades with occasional gravel			Caved-in native soil
35													
									SP-SM	Brown fine to medium sand with silt (medium dense, wet)			
	40		18	18		8							
30													
	45		18	42		9				Becomes dense			
													46.5

Date Excavated	10/31/2018	Total Depth (ft)	11	Logged By	WCW	Excavator		See "Remarks" section for groundwater observed
				Checked By	MSH	Equipment	Takeuchi TB260 Excavator	See "Remarks" section for caving observed

Surface Elevation (ft)	72	Latitude	47.54245	Coordinate System	WA State Plane North
Vertical Datum	NAVD88	Longitude	-122.03392	Horizontal Datum	NAD83 (feet)

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0				TS	Grass and tree roots			
			1 / MC		SM	Brown silty fine to medium sand with organic matter (loose, moist) (fill)	18		Probe depth 8 to 9 inches Large roots from nearby tree
71	1				SM	Brown silty fine to coarse sand with gravel and trace organic matter (loose to medium dense, moist)			
70	2		2						
69	3		3			With oxidation staining			Probe depth 3 to 5 inches
68	4								Moderate caving observed at approximately 4½ feet
67	5				SM	Brown silty fine to medium sand with gravel (medium dense, moist)			
66	6		4 / MC				26		
65	7					Grades with occasional cobbles			
64	8		5						Slight groundwater seepage observed at approximately 8 feet
63	9		6 / MC		SM	Dark brown silty fine to coarse sand with gravel and organic matter (loose to medium dense, wet)	27		
					SP-SM	Brown fine to medium sand with silt and occasional gravel (medium dense, moist)			
62	10		7						
61	11								

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on Topographic Survey. Vertical approximated based on Topographic Survey.

Log of Test Pit TP-1

GeoEngineers

Project: Evergreen Ford Lincoln
Project Location: 22909 SE 66th Street, Issaquah, Washington
Project Number: 23589-001-00

Figure A-7
Sheet 1 of 1

Date Excavated	10/31/2018	Total Depth (ft)	10.5	Logged By WCW Checked By MSH	Excavator Equipment Takeuchi TB260 Excavator	See "Remarks" section for groundwater observed See "Remarks" section for caving observed

Surface Elevation (ft) Vertical Datum	72 NAVD88	Latitude Longitude	47.54205 -122.03374	Coordinate System Horizontal Datum	WA State Plane North NAD83 (feet)

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0				TS	Brown silty fine to medium sand with gravel (topsoil)			Probe depth 6 to 8 inches
71	1		1 MC		SM	Tan-brown silty fine to coarse sand with gravel and occasional cobbles (dense, moist) (fill)	7		
70	1.5				SM	Gray silty fine to medium sand with occasional gravel (medium dense, moist)			
	2		2 MC				16		
69	3		3		SM	Dark brown silty fine to medium sand with gravel and organic matter (medium dense, moist)			
68	3.5				GP-GM	Brown fine gravel with silt and sand (medium dense, moist)			
	4		4						Probe depth 1 to 3 inches
67	4.5				GW-GM	Brown silty fine to coarse gravel with silt, sand and organic matter (medium dense, moist)			
	5		5 SA				7	10	
66	6				SM	Dark gray silty fine to coarse sand with gravel (loose, moist to wet)			Minor caving observed at approximately 6 feet
65	6.5		6 MC				12		
	7		7		SM	Brown silty fine to medium sand (loose to medium dense, moist)			Slight groundwater seepage observed at approximately 7 feet
64	8.5				SM	Dark gray silty fine sand (medium dense, moist)			
63	9		8						
62	10		9		ML	Dark gray-brown sandy silt with gravel (stiff, moist)			

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on Topographic Survey. Vertical approximated based on Topographic Survey.

Date:1/17/19 Path:P:\23\23589001\GINT\2358900100.GPJ DBLibrary/Library:GEOENGINEERS_DF_STD_US_JUNE_2017.GLB/GEI8_TESTPIT_1P_GEOTEC_%F

Log of Test Pit TP-2

GEOENGINEERS

Project: Evergreen Ford Lincoln
Project Location: 22909 SE 66th Street, Issaquah, Washington
Project Number: 23589-001-00

Date Excavated	10/31/2018	Total Depth (ft)	10.5	Logged By WCW / Checked By MSH	Excavator Equipment: Takeuchi TB260 Excavator	See "Remarks" section for groundwater observed / Caving not observed

Surface Elevation (ft)	71	Latitude	47.5416	Coordinate System	WA State Plane North
Vertical Datum	NAVD88	Longitude	-12.03383	Horizontal Datum	NAD83 (feet)

Elevation (feet)	Depth (feet)	Sample Name / Testing	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
71–70.5	0–0.5		TS	Brown silty fine sand with trace organic matter (loose, moist) (fill)			Probe depth 1 to 2 inches
	0.5–1		ML	Gray silt with sand and trace organic matter (medium stiff, moist)			
70	1–3	1, 2	SM	Brown silty fine to coarse sand with gravel (medium dense, moist)			
68	3–4	3	SM	Black-gray silty fine sand (loose to medium dense, moist)			Probe depth 1 to 2 inches
67	4–5	4	SM	Gray silty fine to medium sand with occasional gravel (loose, moist)			
66	5–6.5	5 MC	OL	Dark brown-black organic silt (soft, moist)	67		
64.5	6.5–10	6	SP-SM	Gray fine to coarse sand with silt and gravel (loose to medium dense, moist to wet)			Slight groundwater seepage observed at 8½ feet
61	10–10.5	7	SP-SM	Gray fine to medium sand with silt (loose to medium dense, wet)			

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on Topographic Survey. Vertical approximated based on Topographic Survey.

Log of Test Pit TP-3

GEOENGINEERS

Project: Evergreen Ford Lincoln
Project Location: 22909 SE 66th Street, Issaquah, Washington
Project Number: 23589-001-00

Date Excavated	10/31/2018	Total Depth (ft)	5.5	Logged By	WCW	Excavator		Groundwater not observed
				Checked By	MSH	Equipment	Takeuchi TB260 Excavator	Caving not observed

Surface Elevation (ft)	72	Latitude	47.54207	Coordinate System	WA State Plane North
Vertical Datum	NAVD88	Longitude	-122.03503	Horizontal Datum	NAD83 (feet)

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0				TS	3-4 inch root mat			
					SM	Brown silty fine to coarse sand with gravel (medium dense, moist) (fill)			Probe depth 3 to 6 inches
71	1		1 / SA				14	22	
70	2								Probe depth 2 to 3 inches
69	3								
68	4				ML	Dark gray-black silt with organic matter (soft, moist)			
67	5		2 / SA		SP-SM	Dark gray fine to coarse sand with silt and occasional gravel (loose to medium dense, moist)	17	5	

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on Topographic Survey. Vertical approximated based on Topographic Survey.

Log of Test Pit TP-4

GEOENGINEERS

Project: Evergreen Ford Lincoln
Project Location: 22909 SE 66th Street, Issaquah, Washington
Project Number: 23589-001-00

Figure A-10
Sheet 1 of 1

Date Excavated	10/31/2018	Total Depth (ft)	10.5	Logged By WCW Checked By MSH	Excavator Equipment Takeuchi TB260 Excavator

See "Remarks" section for groundwater observed
See "Remarks" section for caving observed

Surface Elevation (ft) Vertical Datum	73 NAVD88	Latitude Longitude	47.5426 -122.0342	Coordinate System Horizontal Datum	WA State Plane North NAD83 (feet)

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name / Testing	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
73	0			TS	Dark brown silty fine to medium sand with gravel and grass roots (loose, wet) (topsoil)			
	0.5	X	1 / MC	SM	Gray silty fine to coarse sand with gravel (medium dense, moist) (fill)	7		
72	1			SM	Brown silty fine sand with gravel (loose, moist)			
	1.5	X	2					
71	2	X	3 / SA	GW-GM	Reddish brown fine to coarse gravel with silt, sand and occasional cobbles (loose, moist)	6	6	Probe depth 6 to 8 inches Minor caving observed from 3 to 6 feet
69	4			SM	Tan-brown silty fine to medium sand with gravel (loose, moist)			Probe depth 12 to 14 inches
	5.5	X	4					
66	7	X	5	SM	Reddish brown silty fine sand (loose, wet)			Slight groundwater seepage observed at approximately 7½ feet
65	8			ML	Gray silt with occasional sand (soft, wet)			
	8.25			GP-GM	Reddish brown fine to coarse gravel with silt and sand (medium dense, wet)			
	8.5	X	6					Moderate groundwater seepage observed at approximately 9 feet Moderate caving observed at approximately 9 feet
	10	X	7					

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on Topographic Survey. Vertical approximated based on Topographic Survey.

Log of Test Pit TP-5

GeoEngineers

Project: Evergreen Ford Lincoln
Project Location: 22909 SE 66th Street, Issaquah, Washington
Project Number: 23589-001-00

Figure A-11
Sheet 1 of 1

Date Excavated 10/31/2018	Total Depth (ft) 5	Logged By WCW Checked By MSH	Excavator Equipment Takeuchi TB260 Excavator	Groundwater not observed Caving not observed

Surface Elevation (ft) 74 Vertical Datum NAVD88	Latitude 47.54237 Longitude -122.03521	Coordinate System WA State Plane North Horizontal Datum NAD83 (feet)

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0				TS	3-4 inch root mat			
73	1		1 MC		SP-SM	Tan-brown fine to medium sand with silt and occasional gravel (loose to medium dense, moist) (fill)	11		Probe depth 3 to 4 inches
72	2				SM	Blue-gray silty fine to coarse sand with gravel (medium dense, moist)			Probe depth ½ inch
71	3		2						
70	4								
69	5								

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on Topographic Survey. Vertical approximated based on Topographic Survey.

Log of Test Pit TP-6

GeoEngineers

Project: Evergreen Ford Lincoln
Project Location: 22909 SE 66th Street, Issaquah, Washington
Project Number: 23589-001-00

Figure A-12
Sheet 1 of 1

Date Excavated 10/31/2018	Total Depth (ft) 7.5	Logged By WCW Checked By MSH	Excavator Equipment Takeuchi TB260 Excavator	Groundwater not observed Caving not observed

Surface Elevation (ft) 73 Vertical Datum NAVD88	Latitude 47.54177 Longitude -122.03454	Coordinate System WA State Plane North Horizontal Datum NAD83 (feet)

Elevation (feet)	Depth (feet)	SAMPLE Testing Sample	SAMPLE Sample Name Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
					TS	Dark brown silty fine to medium sand (loose, moist) (topsoil)			Probe depth 2 to 3 inches
72	1		1 MC		ML	Tan-brown sandy silt with gravel (stiff, moist) (fill)	18		
71	2		2 MC		SM	Blue-gray silty fine to coarse sand with occasional gravel (medium dense, moist)	18		Probed depth ½ to 1 inch
70	3								
69	4		3						
68	5								
67	6		4 MC		PT	Dark brown organic silt and peat with sand (soft, moist)	76		
66	7								
					SM	Dark gray-brown silty fine to coarse sand with gravel and cobbles (medium dense, moist)			

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on Topographic Survey. Vertical approximated based on Topographic Survey.

Log of Test Pit TP-7

GEOENGINEERS

Project: Evergreen Ford Lincoln
Project Location: 22909 SE 66th Street, Issaquah, Washington
Project Number: 23589-001-00

Date Excavated 10/31/2018	Total Depth (ft) 8.5	Logged By WCW Checked By MSH	Excavator Equipment Takeuchi TB260 Excavator	Groundwater not observed Caving not observed	

Surface Elevation (ft) 73 Vertical Datum NAVD88	Latitude 47.54232 Longitude -122.03484	Coordinate System WA State Plane North Horizontal Datum NAD83 (feet)

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0				TS	3-4 inch root mat			
					SM	Brown silty fine to coarse sand with gravel and occasional cobbles (loose to medium dense, moist) (fill)			Probe depth 3 to 5 inches
72	1		1 / MC				18		
					SM	Dark gray-brown silty fine to coarse sand with gravel (medium dense, moist)			
71	2		2 / SA				10	17	Probe depth ½ inch
70	3								
									Probe depth 3 to 5 inches
69	4								
68	5								
67	6								
66	7								
65	8								

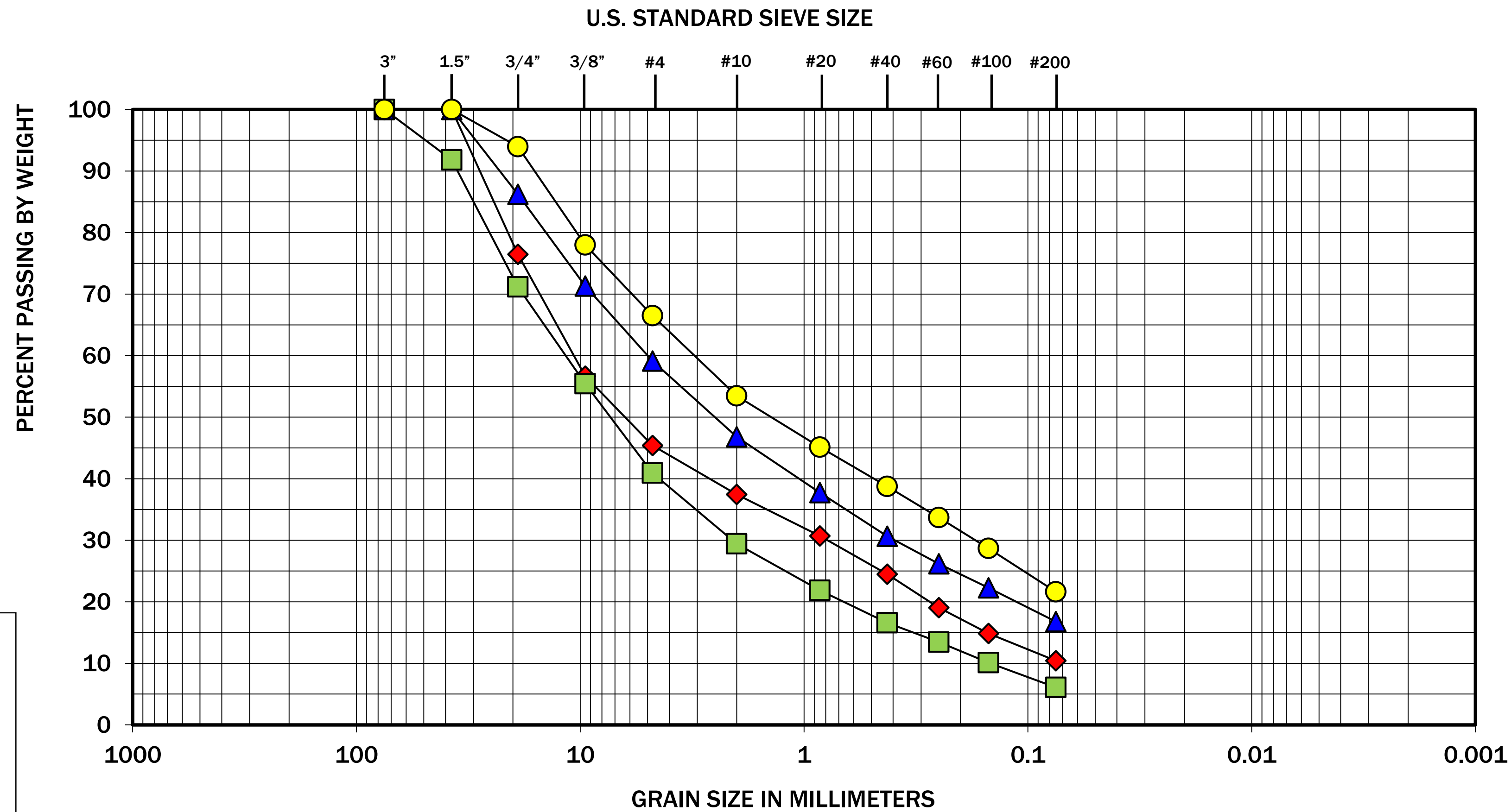
Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on Topographic Survey. Vertical approximated based on Topographic Survey.

Log of Test Pit TP-8

GEOENGINEERS

Project: Evergreen Ford Lincoln
Project Location: 22909 SE 66th Street, Issaquah, Washington
Project Number: 23589-001-00

23589-001-00 Date Exported: 11/14/18



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Symbol	Boring Number	Depth (feet)	Moisture (%)	Soil Description
◆	TP-2	5	7	Fine to coarse gravel with silt and sand (GP-GM)
■	TP-5	3	6	Fine to coarse gravel with silt and sand (GW-GM)
▲	TP-8	2	10	Silty fine to coarse sand with gravel (SM)
●	TP-4	1	14	Silty fine to coarse sand with gravel (SM)

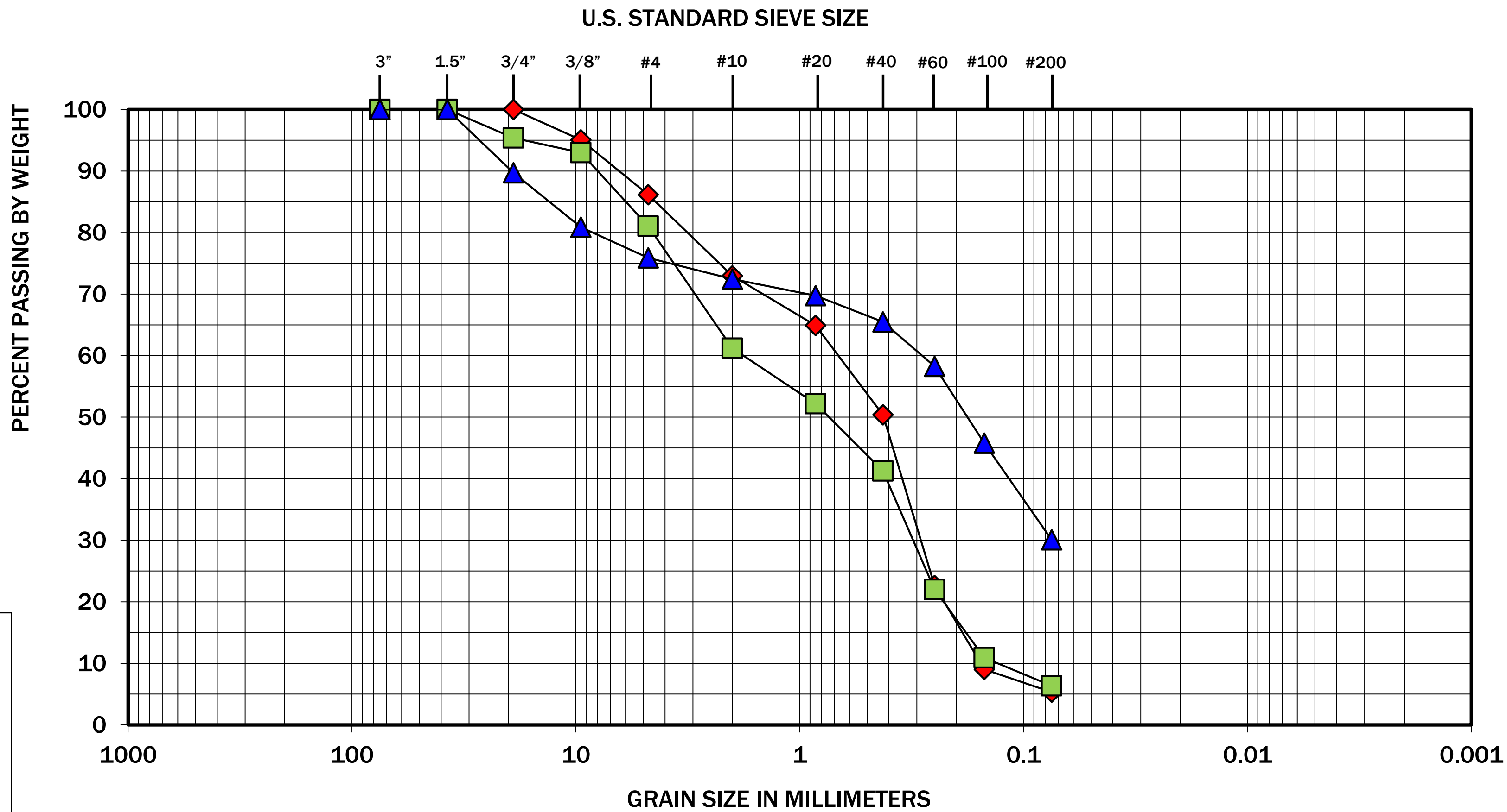
The grain size analysis results were obtained in general accordance with ASTM D 6913. GeoEngineers 17425 NE Union Hill Road Ste 250, Redmond, WA 98052

GeoEngineers

Evergreen Ford Lincoln
Issaquah, Washington

Sieve Analysis Results

Figure A-15



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Symbol	Boring Number	Depth (feet)	Moisture (%)	Soil Description
◆	TP-4	5	17	Fine to coarse sand with silt and occasional gravel (SP-SM)
■	MW-2	5	15	Fine to coarse sand with silt and gravel (SP-SM)
▲	MW-3	5	34	Silty fine sand with gravel (SM)

Note: This report may not be reproduced, except in full, without written approval of GeoEngineers, Inc. Test results are applicable only to the specific sample on which they were performed, and should not be interpreted as representative of any other samples obtained at other times, depths or locations, or generated by separate operations or processes.

The grain size analysis results were obtained in general accordance with ASTM D 6913. GeoEngineers 17425 NE Union Hill Road Ste 250, Redmond, WA 98052

GeoEngineers

Evergreen Ford Lincoln
Issaquah, Washington

Sieve Analysis Results

Figure A-16

APPENDIX B
Previous Explorations

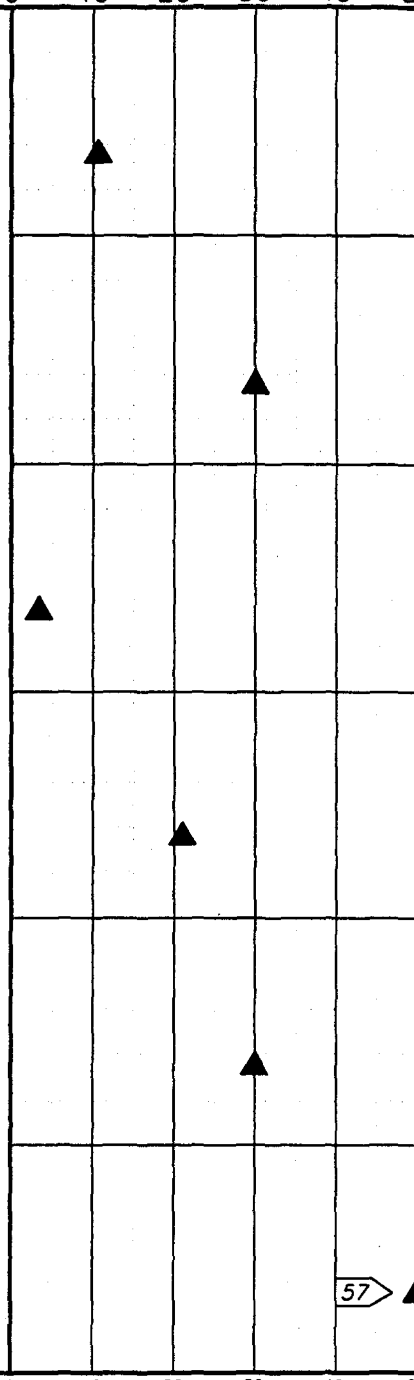
APPENDIX B
PREVIOUS EXPLORATIONS

Included in this section are logs from previous studies completed in the immediate vicinity of the project site.

- The log of one boring (B-1) completed by AGRA Earth & Environmental in 1997 for the AT&T Cellular Tower Site

PROJECT: Issaquah AT&T Cellular Tower Site W.O. 7-91M-11507-0 BORING No. B-1

DEPTH (FEET)	SOIL DESCRIPTION Approximate ground surface elevation: *Unknown*	SAMPLE TYPE	SAMPLE NUMBER	GROUND WATER	STANDARD PENETRATION RESISTANCE ▲ Blows per foot (0 10 20 30 40 50)	Page 1 of 2 TESTING
0	*Soft grading to stiff, wet, brown SILT with trace sand (Fill)*					
		I	*S-1*			
5						
	Medium dense, wet, brown, silty, gravelly SAND					
		I	*S-2*			
10						
				▼ ATD		
	Soft, wet to saturated, gray mottled, SILT	I	*S-3*			
15						
	Medium dense, wet, brown, silty, gravelly SAND					
		I	*S-4*			
20						
		I	*S-5*			
25						
	Very dense, wet, brown, silty SAND	I	*S-6*		57	
	Very dense, wet, brown, silty, sandy GRAVEL					
30						



(continued)

LEGEND

I 2-inch OD split-spoon sample

▼ ATD Groundwater level at time of drilling

0 10 20 30 40 50
MOISTURE CONTENT
Plastic limit Natural Liquid limit

AGRA
Earth & Environmental
11335 N.E. 122nd Way, Suite 100
Kirkland, WA, U.S.A. 98034-6918

Drilling started: *14 February 1997* Drilling completed: *14 February 1997* Logged by: *JRZ*

PROJECT: Issaquah AT&T Cellular Tower Site W.O. 7-91M-11507-0 BORING No. B-1

DEPTH (FEET)	SOIL DESCRIPTION Approximate ground surface elevation: *Unknown*	SAMPLE TYPE	SAMPLE NUMBER	GROUND WATER	STANDARD PENETRATION RESISTANCE ▲ Blows per foot 0 10 20 30 40 50	Page 2 of 2 TESTING
30	*Silty, sandy GRAVEL as above*					
	Very dense, wet, brown SAND	I	S-7A S-7B		60 ▲	
35	*Boring terminated at approximately 34 feet*					
40						
45						
50						
55						
60						

0 10 20 30 40 50
MOISTURE CONTENT
Plastic limit Natural Liquid limit

LEGEND

I 2-inch OD split-spoon sample

ATD Groundwater level at time of drilling

AGRA
Earth & Environmental
11335 N.E. 122nd Way, Suite 100
Kirkland, WA, U.S.A. 98034-6918

Drilling started: *14 February 1997* Drilling completed: *14 February 1997* Logged by: *JRZ*

APPENDIX C
Report Limitations and Guidelines for Use

APPENDIX C
REPORT LIMITATIONS AND GUIDELINES FOR USE[1]

This appendix provides information to help you manage your risks with respect to the use of this report.

Geotechnical Services Are Performed for Specific Purposes, Persons and Projects

This report has been prepared for the exclusive use of Strotkamp Associates and project team members for the Evergreen Ford Lincoln property located in Issaquah, Washington. This report may be made available to prospective contractors for their bidding or estimating purposes, but our report, conclusions and interpretations should not be construed as a warranty of the subsurface conditions. This report is not intended for use by others, and the information contained herein is not applicable to other sites.

GeoEngineers structures our services to meet the specific needs of our clients. For example, a geotechnical or geologic study conducted for a civil engineer or architect may not fulfill the needs of a construction contractor or even another civil engineer or architect that are involved in the same project. Because each geotechnical or geologic study is unique, each geotechnical engineering or geologic report is unique, prepared solely for the specific client and project site. Our report is prepared for the exclusive use of our Client. No other party may rely on the product of our services unless we agree in advance to such reliance in writing. This is to provide our firm with reasonable protection against open-ended liability claims by third parties with which there would otherwise be no contractual limits to their actions. Within the limitations of scope, schedule and budget, our services have been executed in accordance with our Agreement with the Client and generally accepted geotechnical practices in this area at the time this report was prepared. This report should not be applied for any purpose or project except the one originally contemplated.

A Geotechnical Engineering or Geologic Report Is Based on a Unique Set of Project-Specific Factors

This report has been prepared for the Evergreen Ford Lincoln property in Issaquah, Washington. GeoEngineers considered a number of unique, project-specific factors when establishing the scope of services for this project and report. Unless GeoEngineers specifically indicates otherwise, do not rely on this report if it was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

For example, changes that can affect the applicability of this report include those that affect:

- the function of the proposed structure;
- elevation, configuration, location, orientation or weight of the proposed structure;

---

[1] Developed based on material provided by ASFE, Professional Firms Practicing in the Geosciences; www.asfe.org.

- composition of the design team; or
- project ownership.

If important changes are made after the date of this report, GeoEngineers should be given the opportunity to review our interpretations and recommendations and provide written modifications or confirmation, as appropriate.

Subsurface Conditions Can Change

This geotechnical or geologic report is based on conditions that existed at the time the study was performed. The findings and conclusions of this report may be affected by the passage of time, by manmade events such as construction on or adjacent to the site, or by natural events such as floods, earthquakes, slope instability or groundwater fluctuations. Always contact GeoEngineers before applying a report to determine if it remains applicable.

Most Geotechnical and Geologic Findings Are Professional Opinions

Our interpretations of subsurface conditions are based on field observations from widely spaced sampling locations at the site. Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. GeoEngineers reviewed field and laboratory data and then applied our professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ, sometimes significantly, from those indicated in this report. Our report, conclusions and interpretations should not be construed as a warranty of the subsurface conditions.

Geotechnical Engineering Report Recommendations Are Not Final

Do not over-rely on the preliminary construction recommendations included in this report. These recommendations are not final, because they were developed principally from GeoEngineers' professional judgment and opinion. GeoEngineers' recommendations can be finalized only by observing actual subsurface conditions revealed during construction. GeoEngineers cannot assume responsibility or liability for this report's recommendations if we do not perform construction observation.

Sufficient monitoring, testing and consultation by GeoEngineers should be provided during construction to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes should the conditions revealed during the work differ from those anticipated, and to evaluate whether or not earthwork activities are completed in accordance with our recommendations. Retaining GeoEngineers for construction observation for this project is the most effective method of managing the risks associated with unanticipated conditions.

A Geotechnical Engineering or Geologic Report Could Be Subject to Misinterpretation

Misinterpretation of this report by other design team members can result in costly problems. You could lower that risk by having GeoEngineers confer with appropriate members of the design team after submitting the report. Also retain GeoEngineers to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering or geologic report. Reduce that risk by having GeoEngineers participate in pre-bid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Exploration Logs

Geotechnical engineers and geologists prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering or geologic report should never be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, but recognize that separating logs from the report can elevate risk.

Give Contractors a Complete Report and Guidance

Some owners and design professionals believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering or geologic report, but preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with GeoEngineers and/or to conduct additional study to obtain the specific types of information they need or prefer. A pre-bid conference can also be valuable. Be sure contractors have sufficient time to perform additional study. Only then might an owner be in a position to give contractors the best information available, while requiring them to at least share the financial responsibilities stemming from unanticipated conditions. Further, a contingency for unanticipated conditions should be included in your project budget and schedule.

Contractors Are Responsible for Site Safety on Their Own Construction Projects

Our geotechnical recommendations are not intended to direct the contractor's procedures, methods, schedule or management of the work site. The contractor is solely responsible for job site safety and for managing construction operations to minimize risks to on-site personnel and to adjacent properties.

Read These Provisions Closely

Some clients, design professionals and contractors may not recognize that the geoscience practices (geotechnical engineering or geology) are far less exact than other engineering and natural science disciplines. This lack of understanding can create unrealistic expectations that could lead to disappointments, claims and disputes. GeoEngineers includes these explanatory "limitations" provisions in our reports to help reduce such risks. Please confer with GeoEngineers if you are unclear how these "Report Limitations and Guidelines for Use" apply to your project or site.

Geotechnical, Geologic and Environmental Reports Should Not Be Interchanged

The equipment, techniques and personnel used to perform an environmental study differ significantly from those used to perform a geotechnical or geologic study and vice versa. For that reason, a geotechnical engineering or geologic report does not usually relate any environmental findings, conclusions or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. Similarly, environmental reports are not used to address geotechnical or geologic concerns regarding a specific project.

Biological Pollutants

GeoEngineers' Scope of Work specifically excludes the investigation, detection, prevention or assessment of the presence of Biological Pollutants. Accordingly, this report does not include any interpretations, recommendations, findings, or conclusions regarding the detecting, assessing, preventing or abating of Biological Pollutants and no conclusions or inferences should be drawn regarding Biological Pollutants, as they may relate to this project. The term "Biological Pollutants" includes, but is not limited to, molds, fungi, spores, bacteria, and viruses, and/or any of their byproducts.

If Client desires these specialized services, they should be obtained from a consultant who offers services in this specialized field.

GeoEngineers

APPENDIX 6

OPERATIONS AND MAINTENANCE MANUAL

NOT INCLUDED FOR THIS SUBMITTAL

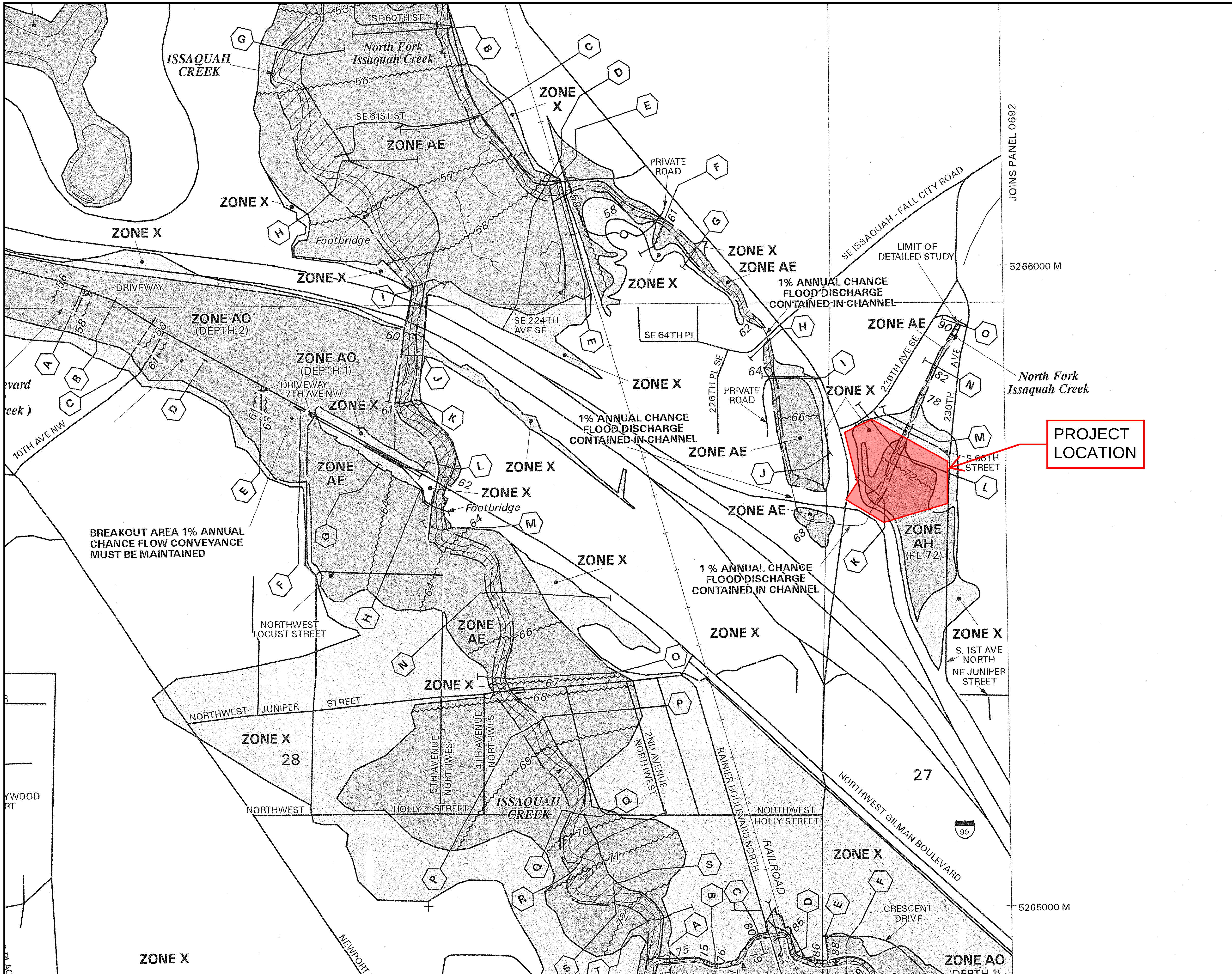
APPENDIX 7

CONSTRUCTION STORMWATER POLLUTION PREVENTION PLAN

NOT INCLUDED FOR THIS SUBMITTAL

APPENDIX 8

FEMA FLOOD INSURANCE MAP



AP SCALE 1" = 500'

500 1000 FEET

METERS

PANEL 0691H

FIRM
FLOOD INSURANCE RATE MAP
KING COUNTY, WASHINGTON AND INCORPORATED AREAS

PANEL 691 OF 1725

(SEE MAP INDEX FOR FIRM PANEL LAYOUT)

CONTAINS:

COMMUNITY	NUMBER	PANEL	SUFFIX
KING COUNTY, UNINCORPORATED AREAS	530071	0691	H
ISSAQUAH, CITY OF	530079	0691	H

Notice to User: The **Map Number** shown below should be used when placing map orders; the **Community Number** shown above should be used on insurance applications for the subject community.

MAP NUMBER
53033C0691H

MAP REVISED:
APRIL 19, 2005

U.S. DEPARTMENT OF HOMELAND SECURITY

Federal Emergency Management Agency

This is an official copy of a portion of the above referenced flood map. It was extracted using F-MIT On-Line. This map does not reflect changes or amendments which may have been made subsequent to the date on the title block. For the latest product information about National Flood Insurance Program flood maps check the FEMA Flood Map Store at www.msc.fema.gov

APPENDIX 9

DESIGN CALCULATIONS AND COMPUTATIONS

WWHM2012

PROJECT REPORT

BASIN 1: FLOW CONTROL

General Model Information

Project Name:	1883.01 Issaquah Evergreen Ford Parking Lot
Site Name:	
Site Address:	
City:	
Report Date:	3/5/2019
Gage:	Seatac
Data Start:	1948/10/01
Data End:	2009/09/30
Timestep:	15 Minute
Precip Scale:	1.333
Version Date:	2018/07/12
Version:	4.2.15

POC Thresholds

Low Flow Threshold for POC1:	50 Percent of the 2 Year
High Flow Threshold for POC1:	50 Year

Landuse Basin Data
Predeveloped Land Use

Basin 1

Bypass:	No
GroundWater:	No
Pervious Land Use	acre
A B, Forest, Flat	1.86
Pervious Total	1.86
Impervious Land Use	acre
Impervious Total	0
Basin Total	1.86

Element Flows To:

Surface	Interflow	Groundwater

Mitigated Land Use

Basin 1

Bypass:	No
GroundWater:	No
Pervious Land Use	acre
A B, Lawn, Flat	0.31
Pervious Total	0.31
Impervious Land Use	acre
ROOF TOPS FLAT	0.03
SIDEWALKS FLAT	0.11
PARKING FLAT	1.41
Impervious Total	1.55
Basin Total	1.86

Element Flows To:

Surface	Interflow	Groundwater
Gravel Trench Bed 1	Gravel Trench Bed 1	

Mitigated Routing

Gravel Trench Bed 1

Bottom Length:	125.00 ft.
Bottom Width:	35.00 ft.
Trench bottom slope 1:	0 To 1
Trench Left side slope 0:	0 To 1
Trench right side slope 2:	0 To 1
Material thickness of first layer:	1
Pour Space of material for first layer:	0.3
Material thickness of second layer:	2
Pour Space of material for second layer:	0.9
Material thickness of third layer:	0
Pour Space of material for third layer:	0
Infiltration On	
Infiltration rate:	5.5
Infiltration safety factor:	1
Total Volume Infiltrated (ac-ft.):	341.193
Total Volume Through Riser (ac-ft.):	0
Total Volume Through Facility (ac-ft.):	341.193
Percent Infiltrated:	100
Total Precip Applied to Facility:	0
Total Evap From Facility:	0

Discharge Structure
Riser Height: 3 ft.
Riser Diameter: 18 in.
Element Flows To:
Outlet 1 Outlet 2

Gravel Trench Bed Hydraulic Table

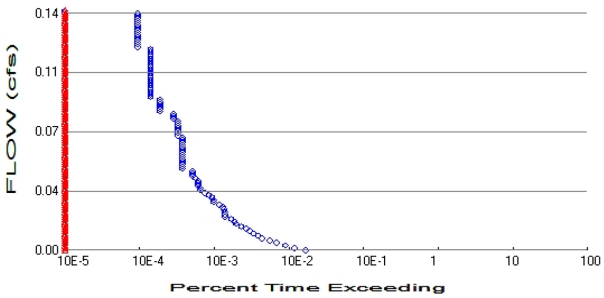
Stage(feet)	**Area(ac.)**	**Volume(ac-ft.)**	**Discharge(cfs)**	**Infilt(cfs)**
0.0000	0.100	0.000	0.000	0.000
0.0444	0.100	0.001	0.000	0.557
0.0889	0.100	0.002	0.000	0.557
0.1333	0.100	0.004	0.000	0.557
0.1778	0.100	0.005	0.000	0.557
0.2222	0.100	0.006	0.000	0.557
0.2667	0.100	0.008	0.000	0.557
0.3111	0.100	0.009	0.000	0.557
0.3556	0.100	0.010	0.000	0.557
0.4000	0.100	0.012	0.000	0.557
0.4444	0.100	0.013	0.000	0.557
0.4889	0.100	0.014	0.000	0.557
0.5333	0.100	0.016	0.000	0.557
0.5778	0.100	0.017	0.000	0.557
0.6222	0.100	0.018	0.000	0.557
0.6667	0.100	0.020	0.000	0.557
0.7111	0.100	0.021	0.000	0.557
0.7556	0.100	0.022	0.000	0.557
0.8000	0.100	0.024	0.000	0.557
0.8444	0.100	0.025	0.000	0.557
0.8889	0.100	0.026	0.000	0.557
0.9333	0.100	0.028	0.000	0.557
0.9778	0.100	0.029	0.000	0.557

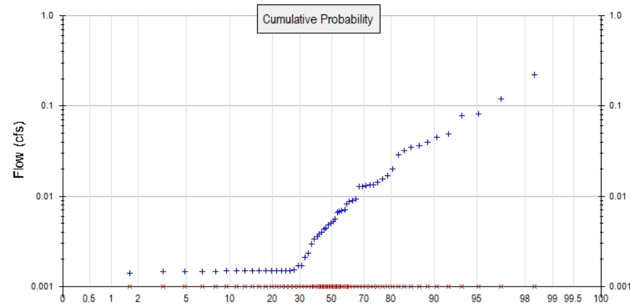
1.0667	0.100	0.037	0.000	0.557
1.1111	0.100	0.041	0.000	0.557
1.1556	0.100	0.045	0.000	0.557
1.2000	0.100	0.049	0.000	0.557
1.2444	0.100	0.053	0.000	0.557
1.2889	0.100	0.057	0.000	0.557
1.3333	0.100	0.061	0.000	0.557
1.3778	0.100	0.065	0.000	0.557
1.4222	0.100	0.069	0.000	0.557
1.4667	0.100	0.073	0.000	0.557
1.5111	0.100	0.077	0.000	0.557
1.5556	0.100	0.081	0.000	0.557
1.6000	0.100	0.085	0.000	0.557
1.6444	0.100	0.089	0.000	0.557
1.6889	0.100	0.093	0.000	0.557
1.7333	0.100	0.097	0.000	0.557
1.7778	0.100	0.101	0.000	0.557
1.8222	0.100	0.105	0.000	0.557
1.8667	0.100	0.109	0.000	0.557
1.9111	0.100	0.113	0.000	0.557
1.9556	0.100	0.117	0.000	0.557
2.0000	0.100	0.121	0.000	0.557
2.0444	0.100	0.125	0.000	0.557
2.0889	0.100	0.129	0.000	0.557
2.1333	0.100	0.133	0.000	0.557
2.1778	0.100	0.137	0.000	0.557
2.2222	0.100	0.141	0.000	0.557
2.2667	0.100	0.146	0.000	0.557
2.3111	0.100	0.150	0.000	0.557
2.3556	0.100	0.154	0.000	0.557
2.4000	0.100	0.158	0.000	0.557
2.4444	0.100	0.162	0.000	0.557
2.4889	0.100	0.166	0.000	0.557
2.5333	0.100	0.170	0.000	0.557
2.5778	0.100	0.174	0.000	0.557
2.6222	0.100	0.178	0.000	0.557
2.6667	0.100	0.182	0.000	0.557
2.7111	0.100	0.186	0.000	0.557
2.7556	0.100	0.190	0.000	0.557
2.8000	0.100	0.194	0.000	0.557
2.8444	0.100	0.198	0.000	0.557
2.8889	0.100	0.202	0.000	0.557
2.9333	0.100	0.206	0.000	0.557
2.9778	0.100	0.210	0.000	0.557
3.0222	0.100	0.214	0.052	0.557
3.0667	0.100	0.219	0.273	0.557
3.1111	0.100	0.223	0.587	0.557
3.1556	0.100	0.228	0.970	0.557
3.2000	0.100	0.232	1.404	0.557
3.2444	0.100	0.237	1.877	0.557
3.2889	0.100	0.241	2.374	0.557
3.3333	0.100	0.246	2.882	0.557
3.3778	0.100	0.250	3.386	0.557
3.4222	0.100	0.254	3.871	0.557
3.4667	0.100	0.259	4.326	0.557
3.5111	0.100	0.263	4.737	0.557
3.5556	0.100	0.268	5.097	0.557
3.6000	0.100	0.272	5.401	0.557

3.6444	0.100	0.277	5.649	0.557
3.6889	0.100	0.281	5.848	0.557
3.7333	0.100	0.286	6.014	0.557
3.7778	0.100	0.290	6.249	0.557
3.8222	0.100	0.295	6.425	0.557
3.8667	0.100	0.299	6.597	0.557
3.9111	0.100	0.304	6.764	0.557
3.9556	0.100	0.308	6.927	0.557
4.0000	0.100	0.312	7.086	0.557

Analysis Results

POC 1





+ Predeveloped x Mitigated

Predeveloped Landuse Totals for POC #1
Total Pervious Area: 1.86
Total Impervious Area: 0

Mitigated Landuse Totals for POC #1
Total Pervious Area: 0.31
Total Impervious Area: 1.55

Flow Frequency Method: Log Pearson Type III 17B

Flow Frequency Return Periods for Predeveloped. POC #1

Return Period	**Flow(cfs)**
2 year	0.005409
5 year	0.018122
10 year	0.036433
25 year	0.080795
50 year	0.139123
100 year	0.23134

Flow Frequency Return Periods for Mitigated. POC #1

Return Period	**Flow(cfs)**
2 year	0
5 year	0
10 year	0
25 year	0
50 year	0
100 year	0

Annual Peaks

Annual Peaks for Predeveloped and Mitigated. POC #1

Year	**Predeveloped**	**Mitigated**
1949	0.004	0.000
1950	0.078	0.000
1951	0.013	0.000
1952	0.004	0.000
1953	0.001	0.000
1954	0.013	0.000
1955	0.002	0.000
1956	0.032	0.000
1957	0.005	0.000

1959	0.005	0.000
1960	0.017	0.000
1961	0.008	0.000
1962	0.001	0.000
1963	0.007	0.000
1964	0.020	0.000
1965	0.004	0.000
1966	0.003	0.000
1967	0.049	0.000
1968	0.013	0.000
1969	0.007	0.000
1970	0.001	0.000
1971	0.007	0.000
1972	0.045	0.000
1973	0.001	0.000
1974	0.005	0.000
1975	0.009	0.000
1976	0.014	0.000
1977	0.001	0.000
1978	0.002	0.000
1979	0.001	0.000
1980	0.001	0.000
1981	0.001	0.000
1982	0.009	0.000
1983	0.002	0.000
1984	0.004	0.000
1985	0.001	0.000
1986	0.002	0.000
1987	0.016	0.000
1988	0.001	0.000
1989	0.001	0.000
1990	0.118	0.000
1991	0.040	0.000
1992	0.001	0.000
1993	0.001	0.000
1994	0.001	0.000
1995	0.029	0.000
1996	0.082	0.000
1997	0.013	0.000
1998	0.007	0.000
1999	0.035	0.000
2000	0.001	0.000
2001	0.001	0.000
2002	0.009	0.000
2003	0.003	0.000
2004	0.004	0.000
2005	0.001	0.000
2006	0.013	0.000
2007	0.221	0.000
2008	0.036	0.000
2009	0.002	0.000

Ranked Annual Peaks

Ranked Annual Peaks for Predeveloped and Mitigated. POC #1

Rank	Predeveloped	Mitigated
1	0.2209	0.0000
2	0.1185	0.0000

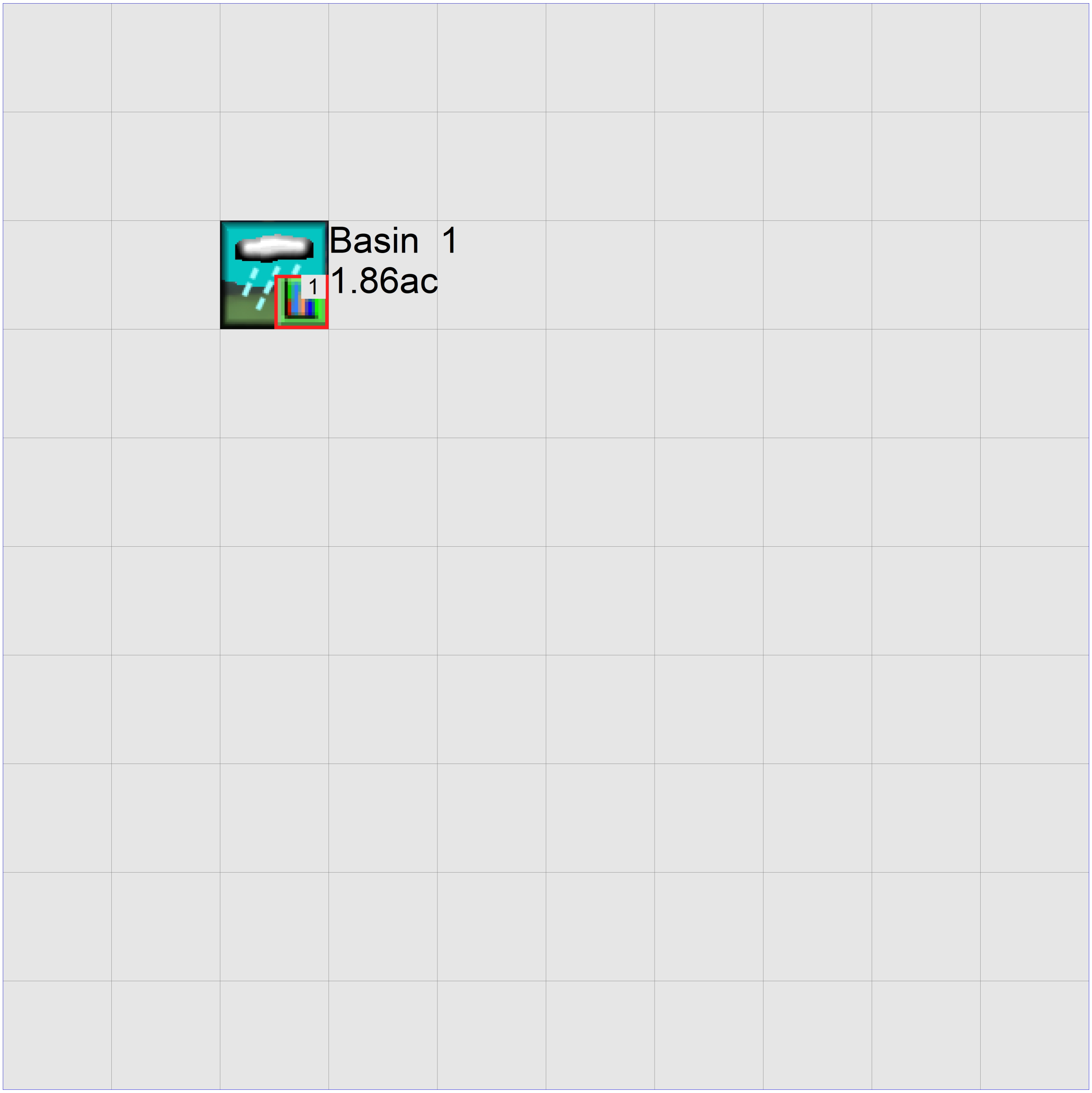
4	0.0781	0.0000
5	0.0485	0.0000
6	0.0454	0.0000
7	0.0398	0.0000
8	0.0360	0.0000
9	0.0352	0.0000
10	0.0323	0.0000
11	0.0289	0.0000
12	0.0202	0.0000
13	0.0168	0.0000
14	0.0156	0.0000
15	0.0142	0.0000
16	0.0135	0.0000
17	0.0133	0.0000
18	0.0130	0.0000
19	0.0129	0.0000
20	0.0129	0.0000
21	0.0093	0.0000
22	0.0089	0.0000
23	0.0087	0.0000
24	0.0082	0.0000
25	0.0071	0.0000
26	0.0069	0.0000
27	0.0068	0.0000
28	0.0066	0.0000
29	0.0055	0.0000
30	0.0052	0.0000
31	0.0050	0.0000
32	0.0048	0.0000
33	0.0044	0.0000
34	0.0043	0.0000
35	0.0040	0.0000
36	0.0038	0.0000
37	0.0035	0.0000
38	0.0034	0.0000
39	0.0030	0.0000
40	0.0024	0.0000
41	0.0021	0.0000
42	0.0017	0.0000
43	0.0017	0.0000
44	0.0015	0.0000
45	0.0015	0.0000
46	0.0015	0.0000
47	0.0015	0.0000
48	0.0015	0.0000
49	0.0015	0.0000
50	0.0015	0.0000
51	0.0015	0.0000
52	0.0015	0.0000
53	0.0015	0.0000
54	0.0015	0.0000
55	0.0015	0.0000
56	0.0015	0.0000
57	0.0015	0.0000
58	0.0015	0.0000
59	0.0015	0.0000
60	0.0014	0.0000
61	0.0010	0.0000

Duration Flows
The Facility PASSED

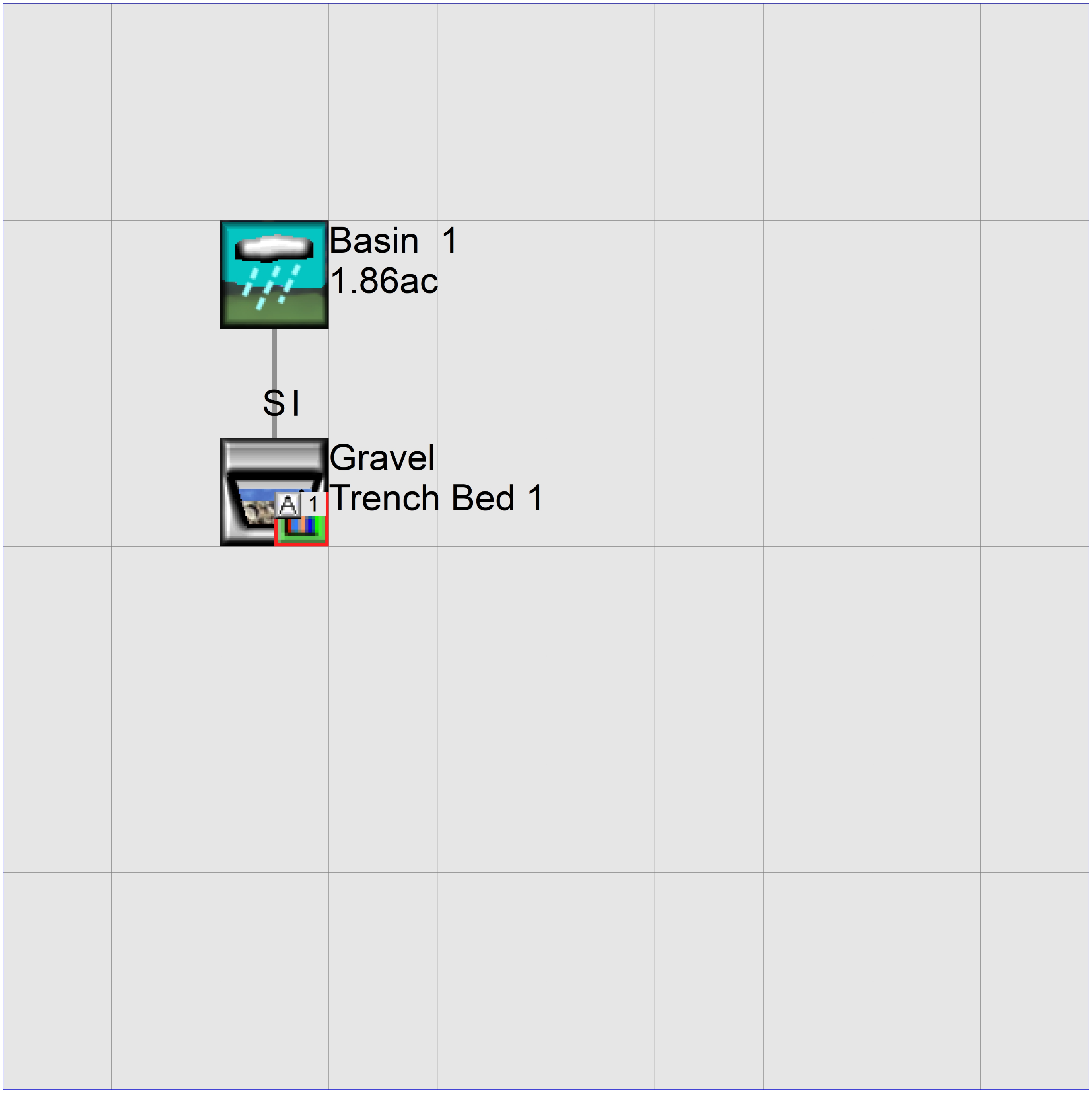
Flow(cfs)	Predev	Mit	Percentage	Pass/Fail
0.0027	361	0	0	Pass
0.0041	255	0	0	Pass
0.0055	193	0	0	Pass
0.0068	144	0	0	Pass
0.0082	119	0	0	Pass
0.0096	92	0	0	Pass
0.0110	82	0	0	Pass
0.0124	72	0	0	Pass
0.0137	64	0	0	Pass
0.0151	57	0	0	Pass
0.0165	49	0	0	Pass
0.0179	42	0	0	Pass
0.0192	40	0	0	Pass
0.0206	36	0	0	Pass
0.0220	30	0	0	Pass
0.0234	30	0	0	Pass
0.0248	30	0	0	Pass
0.0261	29	0	0	Pass
0.0275	28	0	0	Pass
0.0289	25	0	0	Pass
0.0303	21	0	0	Pass
0.0316	21	0	0	Pass
0.0330	20	0	0	Pass
0.0344	18	0	0	Pass
0.0358	16	0	0	Pass
0.0372	14	0	0	Pass
0.0385	14	0	0	Pass
0.0399	13	0	0	Pass
0.0413	13	0	0	Pass
0.0427	13	0	0	Pass
0.0440	12	0	0	Pass
0.0454	11	0	0	Pass
0.0468	11	0	0	Pass
0.0482	11	0	0	Pass
0.0496	8	0	0	Pass
0.0509	8	0	0	Pass
0.0523	8	0	0	Pass
0.0537	8	0	0	Pass
0.0551	8	0	0	Pass
0.0564	8	0	0	Pass
0.0578	8	0	0	Pass
0.0592	8	0	0	Pass
0.0606	8	0	0	Pass
0.0620	8	0	0	Pass
0.0633	8	0	0	Pass
0.0647	8	0	0	Pass
0.0661	8	0	0	Pass
0.0675	8	0	0	Pass
0.0688	7	0	0	Pass
0.0702	7	0	0	Pass
0.0716	7	0	0	Pass
0.0730	7	0	0	Pass

0.0757	7	0	0	Pass
0.0771	7	0	0	Pass
0.0785	6	0	0	Pass
0.0799	6	0	0	Pass
0.0812	6	0	0	Pass
0.0826	4	0	0	Pass
0.0840	4	0	0	Pass
0.0854	4	0	0	Pass
0.0868	4	0	0	Pass
0.0881	4	0	0	Pass
0.0895	4	0	0	Pass
0.0909	3	0	0	Pass
0.0923	3	0	0	Pass
0.0937	3	0	0	Pass
0.0950	3	0	0	Pass
0.0964	3	0	0	Pass
0.0978	3	0	0	Pass
0.0992	3	0	0	Pass
0.1005	3	0	0	Pass
0.1019	3	0	0	Pass
0.1033	3	0	0	Pass
0.1047	3	0	0	Pass
0.1061	3	0	0	Pass
0.1074	3	0	0	Pass
0.1088	3	0	0	Pass
0.1102	3	0	0	Pass
0.1116	3	0	0	Pass
0.1129	3	0	0	Pass
0.1143	3	0	0	Pass
0.1157	3	0	0	Pass
0.1171	3	0	0	Pass
0.1185	3	0	0	Pass
0.1198	2	0	0	Pass
0.1212	2	0	0	Pass
0.1226	2	0	0	Pass
0.1240	2	0	0	Pass
0.1253	2	0	0	Pass
0.1267	2	0	0	Pass
0.1281	2	0	0	Pass
0.1295	2	0	0	Pass
0.1309	2	0	0	Pass
0.1322	2	0	0	Pass
0.1336	2	0	0	Pass
0.1350	2	0	0	Pass
0.1364	2	0	0	Pass
0.1377	2	0	0	Pass
0.1391	2	0	0	Pass

Appendix
Predeveloped Schematic



Mitigated Schematic



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WWHM2012

PROJECT REPORT

BASIN 2:
FLOW
CONTROL

General Model Information

Project Name:	1883.01 Issaquah Evergreen Ford East Side
Site Name:	
Site Address:	
City:	
Report Date:	3/5/2019
Gage:	Seatac
Data Start:	1948/10/01
Data End:	2009/09/30
Timestep:	15 Minute
Precip Scale:	1.333
Version Date:	2018/07/12
Version:	4.2.15

POC Thresholds

Low Flow Threshold for POC1:	50 Percent of the 2 Year
High Flow Threshold for POC1:	50 Year

Landuse Basin Data
Predeveloped Land Use

Basin 1

Bypass:	No
GroundWater:	No
Pervious Land Use	acre
A B, Forest, Flat	0.34
Pervious Total	0.34
Impervious Land Use	acre
Impervious Total	0
Basin Total	0.34

Element Flows To:

Surface	Interflow	Groundwater

Mitigated Land Use

Basin 1

Bypass:	No
GroundWater:	No
Pervious Land Use	acre
Pervious Total	0
Impervious Land Use	acre
ROOF TOPS FLAT	0.13
PARKING FLAT	0.21
Impervious Total	0.34
Basin Total	0.34

Element Flows To:

Surface	Interflow	Groundwater
Gravel Trench Bed 1	Gravel Trench Bed 1	

Mitigated Routing

Gravel Trench Bed 1

Bottom Length:	110.00 ft.
Bottom Width:	15.00 ft.
Trench bottom slope 1:	0 To 1
Trench Left side slope 0:	0 To 1
Trench right side slope 2:	0 To 1
Material thickness of first layer:	1
Pour Space of material for first layer:	0.3
Material thickness of second layer:	2
Pour Space of material for second layer:	0.9
Material thickness of third layer:	0
Pour Space of material for third layer:	0
Infiltration On	
Infiltration rate:	2
Infiltration safety factor:	1
Total Volume Infiltrated (ac-ft.):	74.669
Total Volume Through Riser (ac-ft.):	0
Total Volume Through Facility (ac-ft.):	74.669
Percent Infiltrated:	100
Total Precip Applied to Facility:	0
Total Evap From Facility:	0

Discharge Structure
Riser Height: 3 ft.
Riser Diameter: 18 in.
Element Flows To:
Outlet 1 Outlet 2

Gravel Trench Bed Hydraulic Table

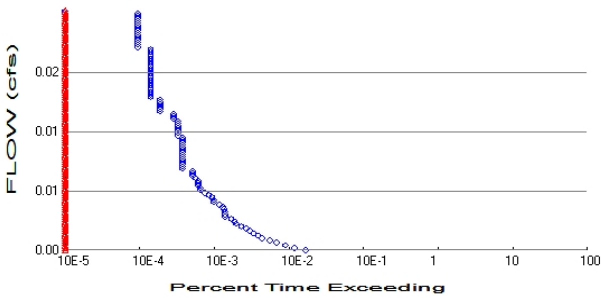
Stage(feet)	**Area(ac.)**	**Volume(ac-ft.)**	**Discharge(cfs)**	**Infilt(cfs)**
0.0000	0.037	0.000	0.000	0.000
0.0444	0.037	0.000	0.000	0.076
0.0889	0.037	0.001	0.000	0.076
0.1333	0.037	0.001	0.000	0.076
0.1778	0.037	0.002	0.000	0.076
0.2222	0.037	0.002	0.000	0.076
0.2667	0.037	0.003	0.000	0.076
0.3111	0.037	0.003	0.000	0.076
0.3556	0.037	0.004	0.000	0.076
0.4000	0.037	0.004	0.000	0.076
0.4444	0.037	0.005	0.000	0.076
0.4889	0.037	0.005	0.000	0.076
0.5333	0.037	0.006	0.000	0.076
0.5778	0.037	0.006	0.000	0.076
0.6222	0.037	0.007	0.000	0.076
0.6667	0.037	0.007	0.000	0.076
0.7111	0.037	0.008	0.000	0.076
0.7556	0.037	0.008	0.000	0.076
0.8000	0.037	0.009	0.000	0.076
0.8444	0.037	0.009	0.000	0.076
0.8889	0.037	0.010	0.000	0.076
0.9333	0.037	0.010	0.000	0.076
0.9778	0.037	0.011	0.000	0.076
1.0222	0.037	0.012	0.000	0.076

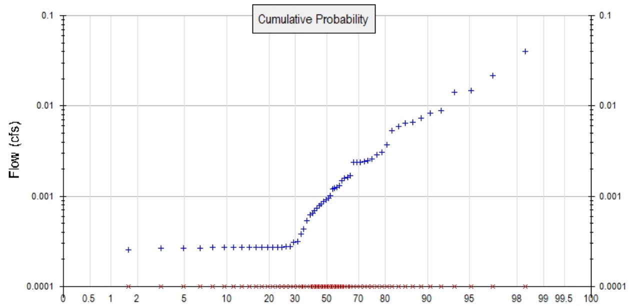
1.0667	0.037	0.014	0.000	0.076
1.1111	0.037	0.015	0.000	0.076
1.1556	0.037	0.017	0.000	0.076
1.2000	0.037	0.018	0.000	0.076
1.2444	0.037	0.020	0.000	0.076
1.2889	0.037	0.021	0.000	0.076
1.3333	0.037	0.023	0.000	0.076
1.3778	0.037	0.024	0.000	0.076
1.4222	0.037	0.026	0.000	0.076
1.4667	0.037	0.027	0.000	0.076
1.5111	0.037	0.029	0.000	0.076
1.5556	0.037	0.030	0.000	0.076
1.6000	0.037	0.032	0.000	0.076
1.6444	0.037	0.033	0.000	0.076
1.6889	0.037	0.035	0.000	0.076
1.7333	0.037	0.036	0.000	0.076
1.7778	0.037	0.038	0.000	0.076
1.8222	0.037	0.039	0.000	0.076
1.8667	0.037	0.041	0.000	0.076
1.9111	0.037	0.042	0.000	0.076
1.9556	0.037	0.044	0.000	0.076
2.0000	0.037	0.046	0.000	0.076
2.0444	0.037	0.047	0.000	0.076
2.0889	0.037	0.049	0.000	0.076
2.1333	0.037	0.050	0.000	0.076
2.1778	0.037	0.052	0.000	0.076
2.2222	0.037	0.053	0.000	0.076
2.2667	0.037	0.055	0.000	0.076
2.3111	0.037	0.056	0.000	0.076
2.3556	0.037	0.058	0.000	0.076
2.4000	0.037	0.059	0.000	0.076
2.4444	0.037	0.061	0.000	0.076
2.4889	0.037	0.062	0.000	0.076
2.5333	0.037	0.064	0.000	0.076
2.5778	0.037	0.065	0.000	0.076
2.6222	0.037	0.067	0.000	0.076
2.6667	0.037	0.068	0.000	0.076
2.7111	0.037	0.070	0.000	0.076
2.7556	0.037	0.071	0.000	0.076
2.8000	0.037	0.073	0.000	0.076
2.8444	0.037	0.074	0.000	0.076
2.8889	0.037	0.076	0.000	0.076
2.9333	0.037	0.077	0.000	0.076
2.9778	0.037	0.079	0.000	0.076
3.0222	0.037	0.081	0.052	0.076
3.0667	0.037	0.082	0.273	0.076
3.1111	0.037	0.084	0.587	0.076
3.1556	0.037	0.086	0.970	0.076
3.2000	0.037	0.087	1.404	0.076
3.2444	0.037	0.089	1.877	0.076
3.2889	0.037	0.091	2.374	0.076
3.3333	0.037	0.092	2.882	0.076
3.3778	0.037	0.094	3.386	0.076
3.4222	0.037	0.096	3.871	0.076
3.4667	0.037	0.097	4.326	0.076
3.5111	0.037	0.099	4.737	0.076
3.5556	0.037	0.101	5.097	0.076
3.6000	0.037	0.102	5.401	0.076

3.6444	0.037	0.104	5.649	0.076
3.6889	0.037	0.106	5.848	0.076
3.7333	0.037	0.107	6.014	0.076
3.7778	0.037	0.109	6.249	0.076
3.8222	0.037	0.111	6.425	0.076
3.8667	0.037	0.113	6.597	0.076
3.9111	0.037	0.114	6.764	0.076
3.9556	0.037	0.116	6.927	0.076
4.0000	0.037	0.118	7.086	0.076

Analysis Results

POC 1





+ Predeveloped   x Mitigated

Predeveloped Landuse Totals for POC #1
Total Pervious Area: 0.34
Total Impervious Area: 0

Mitigated Landuse Totals for POC #1
Total Pervious Area: 0
Total Impervious Area: 0.34

Flow Frequency Method: Log Pearson Type III 17B

Flow Frequency Return Periods for Predeveloped. POC #1

Return Period	**Flow(cfs)**
2 year	0.000989
5 year	0.003313
10 year	0.00666
25 year	0.014769
50 year	0.025431
100 year	0.042288

Flow Frequency Return Periods for Mitigated. POC #1

Return Period	**Flow(cfs)**
2 year	0
5 year	0
10 year	0
25 year	0
50 year	0
100 year	0

Annual Peaks

Annual Peaks for Predeveloped and Mitigated. POC #1

Year	**Predeveloped**	**Mitigated**
1949	0.001	0.000
1950	0.014	0.000
1951	0.002	0.000
1952	0.001	0.000
1953	0.000	0.000
1954	0.002	0.000
1955	0.000	0.000
1956	0.006	0.000
1957	0.001	0.000
1958	0.001	0.000

1959	0.001	0.000
1960	0.003	0.000
1961	0.001	0.000
1962	0.000	0.000
1963	0.001	0.000
1964	0.004	0.000
1965	0.001	0.000
1966	0.001	0.000
1967	0.009	0.000
1968	0.002	0.000
1969	0.001	0.000
1970	0.000	0.000
1971	0.001	0.000
1972	0.008	0.000
1973	0.000	0.000
1974	0.001	0.000
1975	0.002	0.000
1976	0.003	0.000
1977	0.000	0.000
1978	0.000	0.000
1979	0.000	0.000
1980	0.000	0.000
1981	0.000	0.000
1982	0.002	0.000
1983	0.000	0.000
1984	0.001	0.000
1985	0.000	0.000
1986	0.000	0.000
1987	0.003	0.000
1988	0.000	0.000
1989	0.000	0.000
1990	0.022	0.000
1991	0.007	0.000
1992	0.000	0.000
1993	0.000	0.000
1994	0.000	0.000
1995	0.005	0.000
1996	0.015	0.000
1997	0.002	0.000
1998	0.001	0.000
1999	0.006	0.000
2000	0.000	0.000
2001	0.000	0.000
2002	0.002	0.000
2003	0.001	0.000
2004	0.001	0.000
2005	0.000	0.000
2006	0.002	0.000
2007	0.040	0.000
2008	0.007	0.000
2009	0.000	0.000

Ranked Annual Peaks

Ranked Annual Peaks for Predeveloped and Mitigated. POC #1

Rank	**Predeveloped**	**Mitigated**
1	0.0404	0.0000
2	0.0217	0.0000
3	0.0149	0.0000

4	0.0143	0.0000
5	0.0089	0.0000
6	0.0083	0.0000
7	0.0073	0.0000
8	0.0066	0.0000
9	0.0064	0.0000
10	0.0059	0.0000
11	0.0053	0.0000
12	0.0037	0.0000
13	0.0031	0.0000
14	0.0029	0.0000
15	0.0026	0.0000
16	0.0025	0.0000
17	0.0024	0.0000
18	0.0024	0.0000
19	0.0024	0.0000
20	0.0024	0.0000
21	0.0017	0.0000
22	0.0016	0.0000
23	0.0016	0.0000
24	0.0015	0.0000
25	0.0013	0.0000
26	0.0013	0.0000
27	0.0012	0.0000
28	0.0012	0.0000
29	0.0010	0.0000
30	0.0009	0.0000
31	0.0009	0.0000
32	0.0009	0.0000
33	0.0008	0.0000
34	0.0008	0.0000
35	0.0007	0.0000
36	0.0007	0.0000
37	0.0006	0.0000
38	0.0006	0.0000
39	0.0005	0.0000
40	0.0004	0.0000
41	0.0004	0.0000
42	0.0003	0.0000
43	0.0003	0.0000
44	0.0003	0.0000
45	0.0003	0.0000
46	0.0003	0.0000
47	0.0003	0.0000
48	0.0003	0.0000
49	0.0003	0.0000
50	0.0003	0.0000
51	0.0003	0.0000
52	0.0003	0.0000
53	0.0003	0.0000
54	0.0003	0.0000
55	0.0003	0.0000
56	0.0003	0.0000
57	0.0003	0.0000
58	0.0003	0.0000
59	0.0003	0.0000
60	0.0003	0.0000
61	0.0002	0.0000

Duration Flows

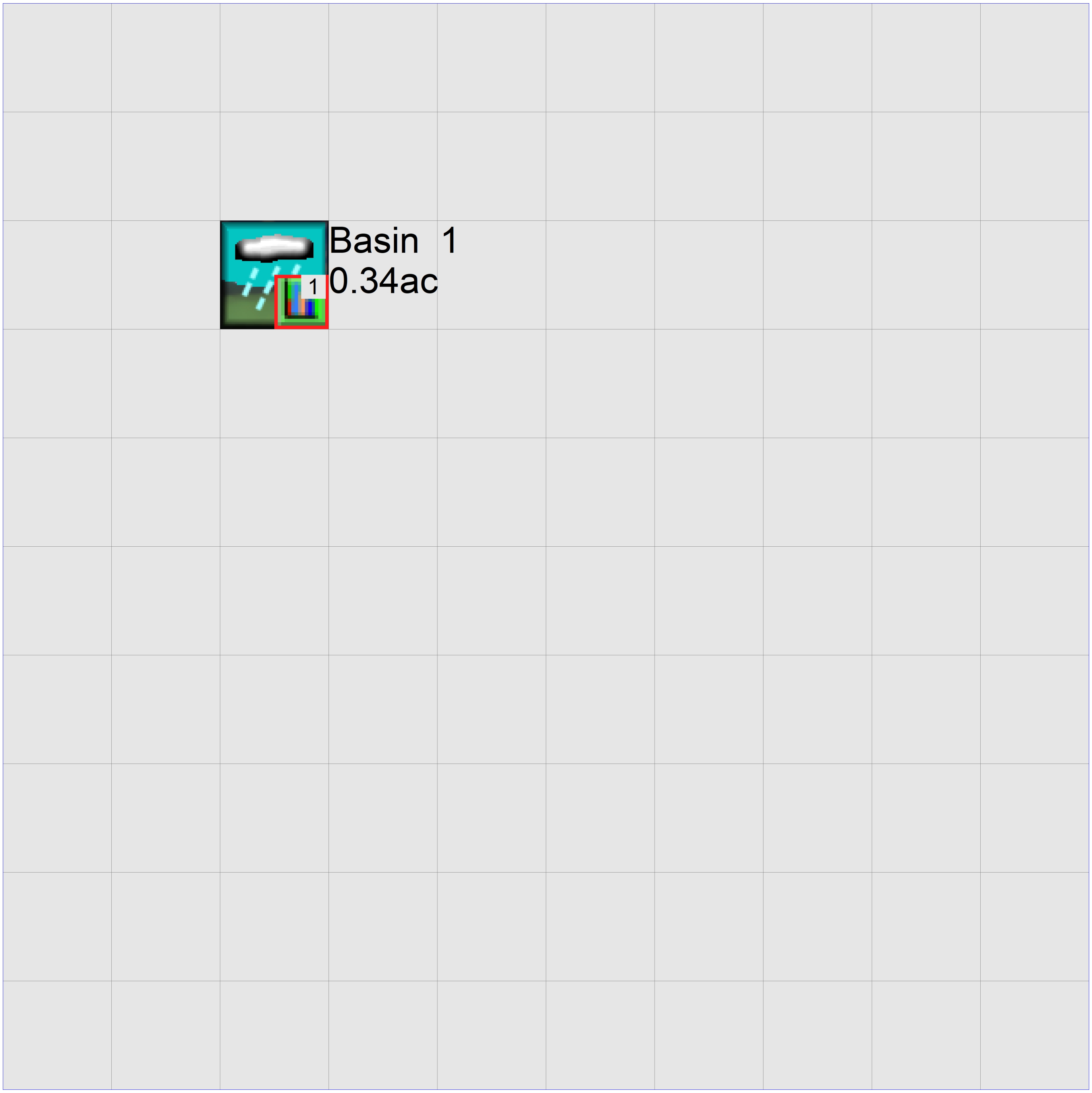
The Facility PASSED

Flow(cfs)	Predev	Mit	Percentage	Pass/Fail
0.0005	361	0	0	Pass
0.0007	255	0	0	Pass
0.0010	193	0	0	Pass
0.0013	144	0	0	Pass
0.0015	119	0	0	Pass
0.0018	93	0	0	Pass
0.0020	82	0	0	Pass
0.0023	72	0	0	Pass
0.0025	64	0	0	Pass
0.0028	57	0	0	Pass
0.0030	49	0	0	Pass
0.0033	42	0	0	Pass
0.0035	40	0	0	Pass
0.0038	36	0	0	Pass
0.0040	30	0	0	Pass
0.0043	30	0	0	Pass
0.0045	30	0	0	Pass
0.0048	29	0	0	Pass
0.0050	28	0	0	Pass
0.0053	25	0	0	Pass
0.0055	21	0	0	Pass
0.0058	21	0	0	Pass
0.0060	20	0	0	Pass
0.0063	18	0	0	Pass
0.0065	16	0	0	Pass
0.0068	14	0	0	Pass
0.0070	14	0	0	Pass
0.0073	13	0	0	Pass
0.0075	13	0	0	Pass
0.0078	13	0	0	Pass
0.0081	12	0	0	Pass
0.0083	11	0	0	Pass
0.0086	11	0	0	Pass
0.0088	11	0	0	Pass
0.0091	8	0	0	Pass
0.0093	8	0	0	Pass
0.0096	8	0	0	Pass
0.0098	8	0	0	Pass
0.0101	8	0	0	Pass
0.0103	8	0	0	Pass
0.0106	8	0	0	Pass
0.0108	8	0	0	Pass
0.0111	8	0	0	Pass
0.0113	8	0	0	Pass
0.0116	8	0	0	Pass
0.0118	8	0	0	Pass
0.0121	8	0	0	Pass
0.0123	8	0	0	Pass
0.0126	7	0	0	Pass
0.0128	7	0	0	Pass
0.0131	7	0	0	Pass
0.0133	7	0	0	Pass
0.0136	7	0	0	Pass

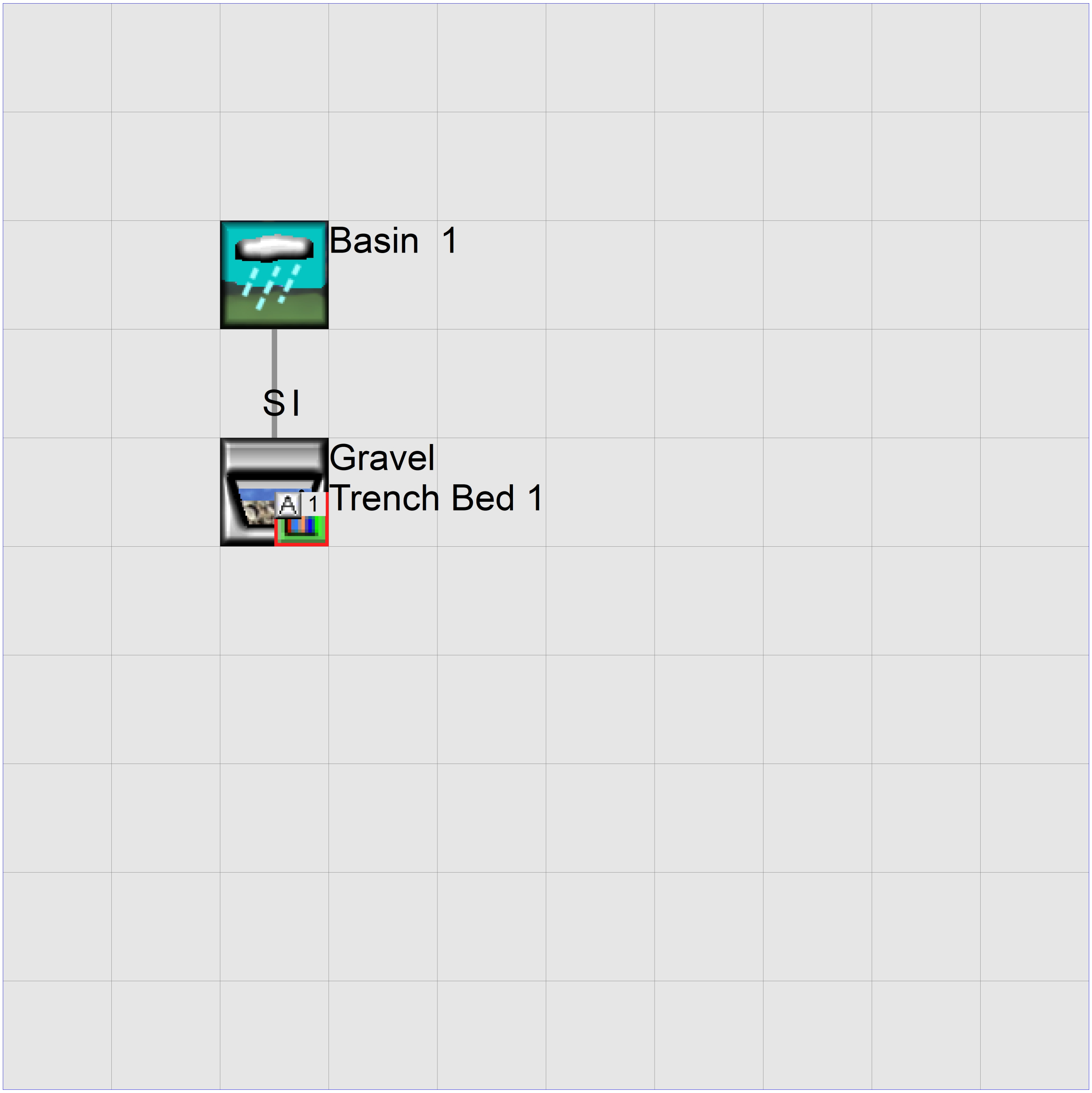
0.0138	7	0	0	Pass
0.0141	7	0	0	Pass
0.0143	6	0	0	Pass
0.0146	6	0	0	Pass
0.0149	6	0	0	Pass
0.0151	4	0	0	Pass
0.0154	4	0	0	Pass
0.0156	4	0	0	Pass
0.0159	4	0	0	Pass
0.0161	4	0	0	Pass
0.0164	4	0	0	Pass
0.0166	3	0	0	Pass
0.0169	3	0	0	Pass
0.0171	3	0	0	Pass
0.0174	3	0	0	Pass
0.0176	3	0	0	Pass
0.0179	3	0	0	Pass
0.0181	3	0	0	Pass
0.0184	3	0	0	Pass
0.0186	3	0	0	Pass
0.0189	3	0	0	Pass
0.0191	3	0	0	Pass
0.0194	3	0	0	Pass
0.0196	3	0	0	Pass
0.0199	3	0	0	Pass
0.0201	3	0	0	Pass
0.0204	3	0	0	Pass
0.0206	3	0	0	Pass
0.0209	3	0	0	Pass
0.0211	3	0	0	Pass
0.0214	3	0	0	Pass
0.0217	3	0	0	Pass
0.0219	2	0	0	Pass
0.0222	2	0	0	Pass
0.0224	2	0	0	Pass
0.0227	2	0	0	Pass
0.0229	2	0	0	Pass
0.0232	2	0	0	Pass
0.0234	2	0	0	Pass
0.0237	2	0	0	Pass
0.0239	2	0	0	Pass
0.0242	2	0	0	Pass
0.0244	2	0	0	Pass
0.0247	2	0	0	Pass
0.0249	2	0	0	Pass
0.0252	2	0	0	Pass
0.0254	2	0	0	Pass

Appendix

Predeveloped Schematic



Mitigated Schematic



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WWHM2012

PROJECT REPORT

BASIN 3:
FLOW
CONTROL
AND
TREATMENT

General Model Information

Project Name:	1883.01 Issaquah Evergreen Ford Roof
Site Name:	
Site Address:	
City:	
Report Date:	3/5/2019
Gage:	Seatac
Data Start:	1948/10/01
Data End:	2009/09/30
Timestep:	15 Minute
Precip Scale:	1.333
Version Date:	2018/07/12
Version:	4.2.15

POC Thresholds

Low Flow Threshold for POC1:	50 Percent of the 2 Year
High Flow Threshold for POC1:	50 Year

Landuse Basin Data
Predeveloped Land Use

Basin 1

Bypass:	No
GroundWater:	No
Pervious Land Use	acre
A B, Forest, Flat	1.38
Pervious Total	1.38
Impervious Land Use	acre
Impervious Total	0
Basin Total	1.38

Element Flows To:

Surface	Interflow	Groundwater

Mitigated Land Use

Basin 1

Bypass:	No
GroundWater:	No
Pervious Land Use	acre
A B, Lawn, Flat	0.25
Pervious Total	0.25
Impervious Land Use	acre
ROOF TOPS FLAT	0.91
SIDEWALKS FLAT	0.09
PARKING FLAT	0.09
Impervious Total	1.09
Basin Total	1.34

Element Flows To:

Surface	Interflow	Groundwater
Gravel Trench Bed 1	Gravel Trench Bed 1	

Mitigated Routing

Gravel Trench Bed 1

Bottom Length:	100.00 ft.
Bottom Width:	37.60 ft.
Trench bottom slope 1:	3 To 1
Trench Left side slope 0:	3 To 1
Trench right side slope 2:	3 To 1
Material thickness of first layer:	1
Pour Space of material for first layer:	0.33
Material thickness of second layer:	1.5
Pour Space of material for second layer:	0.4
Material thickness of third layer:	0
Pour Space of material for third layer:	0
Infiltration On	
Infiltration rate:	2
Infiltration safety factor:	1
Wetted surface area On	
Total Volume Infiltrated (ac-ft.):	261.06
Total Volume Through Riser (ac-ft.):	0.012
Total Volume Through Facility (ac-ft.):	261.072
Percent Infiltrated:	100
Total Precip Applied to Facility:	20.966
Total Evap From Facility:	0

Discharge Structure

Riser Height:	3.5 ft.
Riser Diameter:	12 in.

Element Flows To:

Outlet 1 Outlet 2

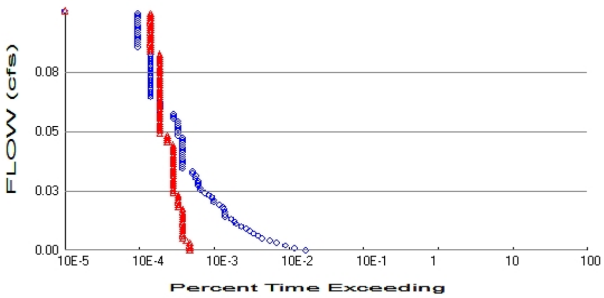
Gravel Trench Bed Hydraulic Table

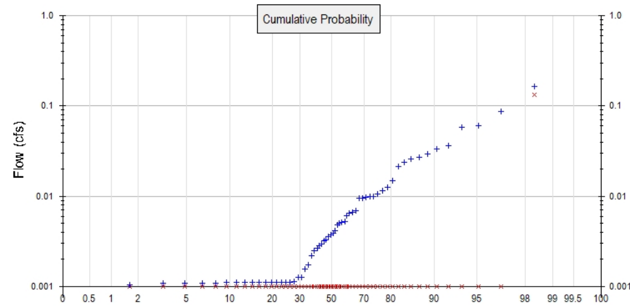
Stage(feet)	Area(ac.)	Volume(ac-ft.)	Discharge(cfs)	Infilt(cfs)
0.0000	0.086	0.000	0.000	0.000
0.0444	0.087	0.001	0.000	0.175
0.0889	0.088	0.002	0.000	0.177
0.1333	0.088	0.003	0.000	0.179
0.1778	0.089	0.005	0.000	0.180
0.2222	0.090	0.006	0.000	0.182
0.2667	0.091	0.007	0.000	0.184
0.3111	0.092	0.009	0.000	0.186
0.3556	0.093	0.010	0.000	0.187
0.4000	0.094	0.011	0.000	0.189
0.4444	0.094	0.013	0.000	0.191
0.4889	0.095	0.014	0.000	0.193
0.5333	0.096	0.016	0.000	0.194
0.5778	0.097	0.017	0.000	0.196
0.6222	0.098	0.019	0.000	0.198
0.6667	0.099	0.020	0.000	0.200
0.7111	0.100	0.021	0.000	0.202
0.7556	0.101	0.023	0.000	0.203
0.8000	0.102	0.024	0.000	0.205
0.8444	0.102	0.026	0.000	0.207
0.8889	0.103	0.027	0.000	0.209
0.9333	0.104	0.029	0.000	0.211
0.9778	0.105	0.030	0.000	0.213

1.0222	0.106	0.032	0.000	0.214
1.0667	0.107	0.034	0.000	0.216
1.1111	0.108	0.036	0.000	0.218
1.1556	0.109	0.038	0.000	0.220
1.2000	0.110	0.040	0.000	0.222
1.2444	0.111	0.042	0.000	0.224
1.2889	0.112	0.044	0.000	0.226
1.3333	0.113	0.046	0.000	0.228
1.3778	0.114	0.048	0.000	0.229
1.4222	0.114	0.050	0.000	0.231
1.4667	0.115	0.052	0.000	0.233
1.5111	0.116	0.054	0.000	0.235
1.5556	0.117	0.056	0.000	0.237
1.6000	0.118	0.058	0.000	0.239
1.6444	0.119	0.061	0.000	0.241
1.6889	0.120	0.063	0.000	0.243
1.7333	0.121	0.065	0.000	0.245
1.7778	0.122	0.067	0.000	0.247
1.8222	0.123	0.069	0.000	0.249
1.8667	0.124	0.071	0.000	0.251
1.9111	0.125	0.074	0.000	0.253
1.9556	0.126	0.076	0.000	0.255
2.0000	0.127	0.078	0.000	0.257
2.0444	0.128	0.080	0.000	0.259
2.0889	0.129	0.083	0.000	0.261
2.1333	0.130	0.085	0.000	0.263
2.1778	0.131	0.087	0.000	0.265
2.2222	0.132	0.090	0.000	0.267
2.2667	0.133	0.092	0.000	0.269
2.3111	0.134	0.094	0.000	0.271
2.3556	0.135	0.097	0.000	0.273
2.4000	0.136	0.099	0.000	0.275
2.4444	0.137	0.102	0.000	0.277
2.4889	0.138	0.104	0.000	0.279
2.5333	0.139	0.110	0.000	0.281
2.5778	0.140	0.117	0.000	0.283
2.6222	0.141	0.123	0.000	0.285
2.6667	0.142	0.129	0.000	0.287
2.7111	0.143	0.135	0.000	0.289
2.7556	0.144	0.142	0.000	0.292
2.8000	0.145	0.148	0.000	0.294
2.8444	0.146	0.155	0.000	0.296
2.8889	0.148	0.161	0.000	0.298
2.9333	0.149	0.168	0.000	0.300
2.9778	0.150	0.175	0.000	0.302
3.0222	0.151	0.181	0.000	0.304
3.0667	0.152	0.188	0.000	0.307
3.1111	0.153	0.195	0.000	0.309
3.1556	0.154	0.202	0.000	0.311
3.2000	0.155	0.209	0.000	0.313
3.2444	0.156	0.216	0.000	0.315
3.2889	0.157	0.223	0.000	0.317
3.3333	0.158	0.230	0.000	0.320
3.3778	0.159	0.237	0.000	0.322
3.4222	0.160	0.244	0.000	0.324
3.4667	0.162	0.251	0.000	0.326
3.5111	0.163	0.258	0.012	0.328
3.5556	0.164	0.265	0.138	0.331

3.6000	0.165	0.273	0.333	0.333
3.6444	0.166	0.280	0.572	0.335
3.6889	0.167	0.288	0.838	0.337
3.7333	0.168	0.295	1.115	0.340
3.7778	0.169	0.303	1.383	0.342
3.8222	0.170	0.310	1.627	0.344
3.8667	0.172	0.318	1.834	0.346
3.9111	0.173	0.325	1.996	0.349
3.9556	0.174	0.333	2.114	0.351
4.0000	0.175	0.341	2.203	0.353

Analysis Results
POC 1





+ Predeveloped   x Mitigated

Predeveloped Landuse Totals for POC #1
Total Pervious Area: 1.38
Total Impervious Area: 0

Mitigated Landuse Totals for POC #1
Total Pervious Area: 0.25
Total Impervious Area: 1.09

Flow Frequency Method: Log Pearson Type III 17B

Flow Frequency Return Periods for Predeveloped. POC #1

Return Period	**Flow(cfs)**
2 year	0.004013
5 year	0.013445
10 year	0.027031
25 year	0.059945
50 year	0.10322
100 year	0.171639

Flow Frequency Return Periods for Mitigated. POC #1

Return Period	**Flow(cfs)**
2 year	0
5 year	0
10 year	0
25 year	0
50 year	0
100 year	0

Annual Peaks
Annual Peaks for Predeveloped and Mitigated. POC #1

Year	**Predeveloped**	**Mitigated**
1949	0.003	0.000
1950	0.058	0.000
1951	0.010	0.000
1952	0.003	0.000
1953	0.001	0.000
1954	0.010	0.000
1955	0.002	0.000
1956	0.024	0.000
1957	0.004	0.000
1958	0.004	0.000

1959	0.004	0.000
1960	0.012	0.000
1961	0.006	0.000
1962	0.001	0.000
1963	0.005	0.000
1964	0.015	0.000
1965	0.003	0.000
1966	0.002	0.000
1967	0.036	0.000
1968	0.010	0.000
1969	0.005	0.000
1970	0.001	0.000
1971	0.005	0.000
1972	0.034	0.000
1973	0.001	0.000
1974	0.004	0.000
1975	0.007	0.000
1976	0.011	0.000
1977	0.001	0.000
1978	0.001	0.000
1979	0.001	0.000
1980	0.001	0.000
1981	0.001	0.000
1982	0.006	0.000
1983	0.001	0.000
1984	0.003	0.000
1985	0.001	0.000
1986	0.001	0.000
1987	0.012	0.000
1988	0.001	0.000
1989	0.001	0.000
1990	0.088	0.000
1991	0.030	0.000
1992	0.001	0.000
1993	0.001	0.000
1994	0.001	0.000
1995	0.021	0.000
1996	0.061	0.000
1997	0.010	0.000
1998	0.005	0.000
1999	0.026	0.000
2000	0.001	0.000
2001	0.001	0.000
2002	0.007	0.000
2003	0.003	0.000
2004	0.003	0.133
2005	0.001	0.000
2006	0.010	0.000
2007	0.164	0.000
2008	0.027	0.000
2009	0.002	0.000

Ranked Annual Peaks

Ranked Annual Peaks for Predeveloped and Mitigated. POC #1

Rank	Predeveloped	Mitigated
1	0.1639	0.1332
2	0.0879	0.0000
3	0.0606	0.0000

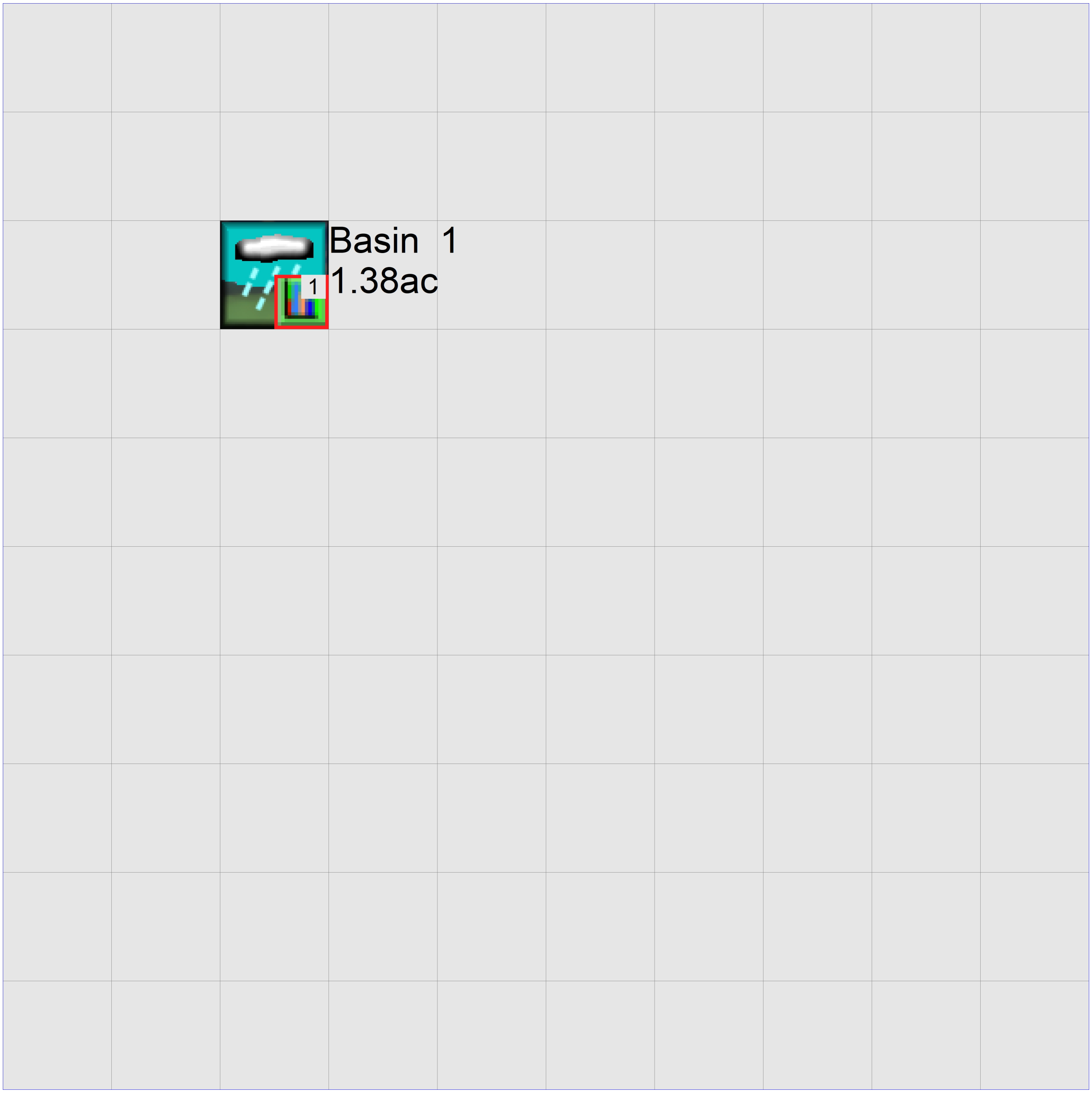
4	0.0579	0.0000
5	0.0360	0.0000
6	0.0337	0.0000
7	0.0295	0.0000
8	0.0267	0.0000
9	0.0261	0.0000
10	0.0240	0.0000
11	0.0215	0.0000
12	0.0150	0.0000
13	0.0125	0.0000
14	0.0116	0.0000
15	0.0106	0.0000
16	0.0100	0.0000
17	0.0098	0.0000
18	0.0096	0.0000
19	0.0096	0.0000
20	0.0096	0.0000
21	0.0069	0.0000
22	0.0066	0.0000
23	0.0065	0.0000
24	0.0061	0.0000
25	0.0053	0.0000
26	0.0051	0.0000
27	0.0050	0.0000
28	0.0049	0.0000
29	0.0041	0.0000
30	0.0039	0.0000
31	0.0037	0.0000
32	0.0035	0.0000
33	0.0033	0.0000
34	0.0032	0.0000
35	0.0030	0.0000
36	0.0028	0.0000
37	0.0026	0.0000
38	0.0025	0.0000
39	0.0022	0.0000
40	0.0017	0.0000
41	0.0015	0.0000
42	0.0013	0.0000
43	0.0013	0.0000
44	0.0011	0.0000
45	0.0011	0.0000
46	0.0011	0.0000
47	0.0011	0.0000
48	0.0011	0.0000
49	0.0011	0.0000
50	0.0011	0.0000
51	0.0011	0.0000
52	0.0011	0.0000
53	0.0011	0.0000
54	0.0011	0.0000
55	0.0011	0.0000
56	0.0011	0.0000
57	0.0011	0.0000
58	0.0011	0.0000
59	0.0011	0.0000
60	0.0010	0.0000
61	0.0007	0.0000

Duration Flows

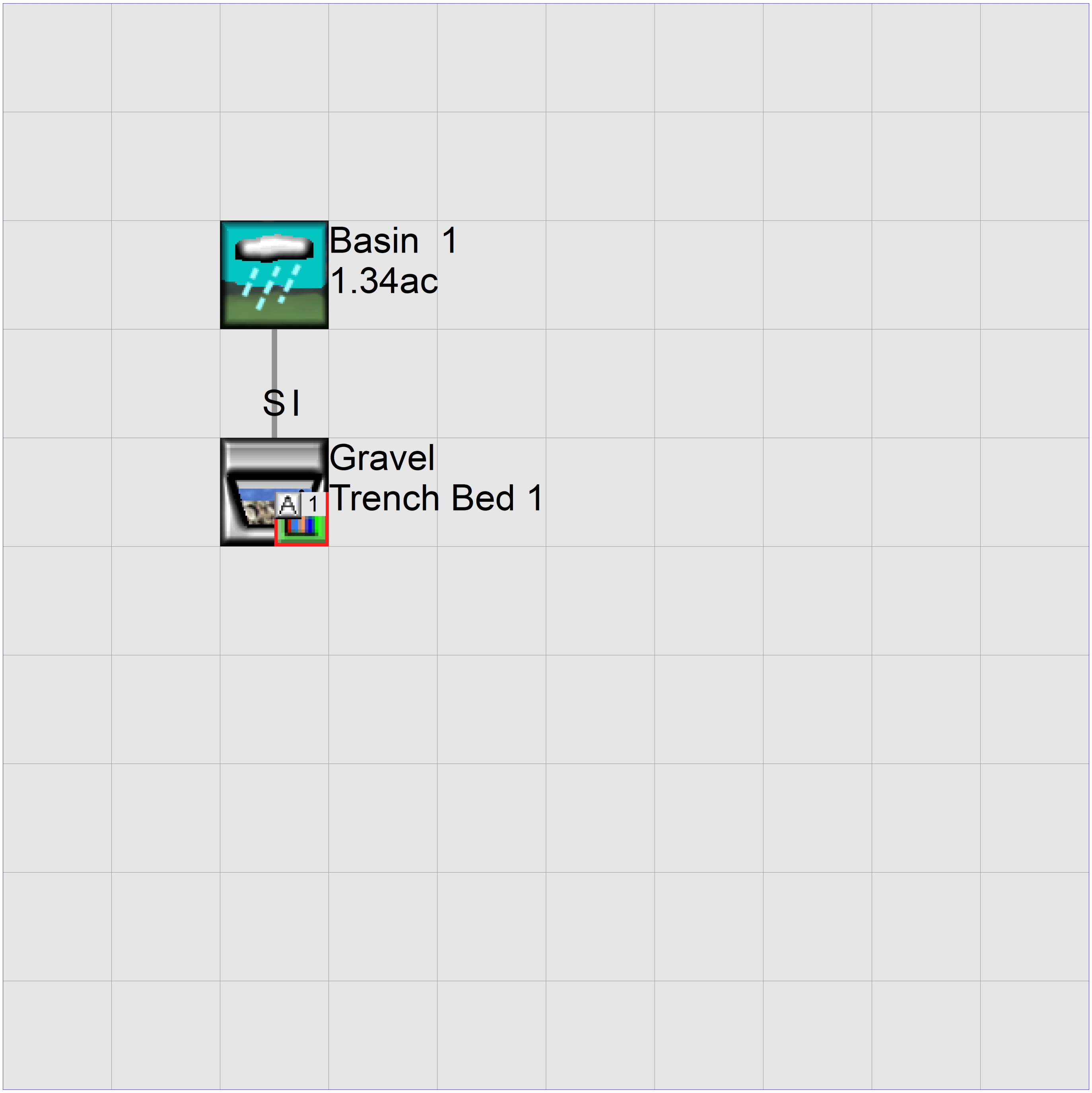
Flow(cfs)	Predev	Mit	Percentage	Pass/Fail
0.0020	361	10	2	Pass
0.0030	255	10	3	Pass
0.0041	193	10	5	Pass
0.0051	144	10	6	Pass
0.0061	119	9	7	Pass
0.0071	92	8	8	Pass
0.0081	82	8	9	Pass
0.0092	72	8	11	Pass
0.0102	64	8	12	Pass
0.0112	57	8	14	Pass
0.0122	49	8	16	Pass
0.0133	42	8	19	Pass
0.0143	40	8	20	Pass
0.0153	36	8	22	Pass
0.0163	30	8	26	Pass
0.0173	30	8	26	Pass
0.0184	30	8	26	Pass
0.0194	29	8	27	Pass
0.0204	28	7	25	Pass
0.0214	25	7	28	Pass
0.0225	21	7	33	Pass
0.0235	21	7	33	Pass
0.0245	20	7	35	Pass
0.0255	18	7	38	Pass
0.0265	16	6	37	Pass
0.0276	14	6	42	Pass
0.0286	14	6	42	Pass
0.0296	13	6	46	Pass
0.0306	13	6	46	Pass
0.0317	13	6	46	Pass
0.0327	12	6	50	Pass
0.0337	12	6	50	Pass
0.0347	11	6	54	Pass
0.0357	11	6	54	Pass
0.0368	8	6	75	Pass
0.0378	8	6	75	Pass
0.0388	8	6	75	Pass
0.0398	8	6	75	Pass
0.0409	8	6	75	Pass
0.0419	8	6	75	Pass
0.0429	8	6	75	Pass
0.0439	8	6	75	Pass
0.0449	8	6	75	Pass
0.0460	8	6	75	Pass
0.0470	8	6	75	Pass
0.0480	8	5	62	Pass
0.0490	8	5	62	Pass
0.0501	8	5	62	Pass
0.0511	7	5	71	Pass
0.0521	7	4	57	Pass
0.0531	7	4	57	Pass
0.0541	7	4	57	Pass
0.0552	7	4	57	Pass
0.0562	7	4	57	Pass

Appendix

Predeveloped Schematic



Mitigated Schematic



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PROJECT REPORT

BASIN 1:
TREATMENT

General Model Information

Project Name:	1883.01 Issaquah Basin 1 Treatment
Site Name:	
Site Address:	
City:	
Report Date:	3/5/2019
Gage:	Seatac
Data Start:	1948/10/01
Data End:	2009/09/30
Timestep:	15 Minute
Precip Scale:	1.333
Version Date:	2018/07/12
Version:	4.2.15

POC Thresholds

Low Flow Threshold for POC1:	50 Percent of the 2 Year
High Flow Threshold for POC1:	50 Year

Landuse Basin Data
Predeveloped Land Use

Basin 1

Bypass:	No
GroundWater:	No
Pervious Land Use	acre
A B, Forest, Flat	1.86
Pervious Total	1.86
Impervious Land Use	acre
Impervious Total	0
Basin Total	1.86

Element Flows To:

Surface	Interflow	Groundwater

Mitigated Land Use

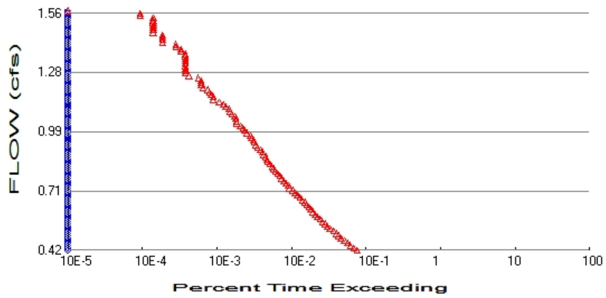
Basin 1

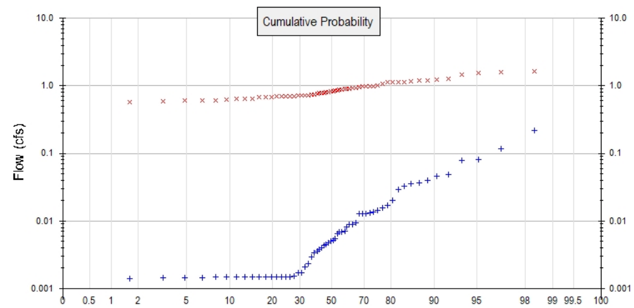
Bypass:	No
GroundWater:	No
Pervious Land Use	acre
A B, Lawn, Flat	0.31
Pervious Total	0.31
Impervious Land Use	acre
ROOF TOPS FLAT	0.03
SIDEWALKS FLAT	0.11
PARKING FLAT	1.41
Impervious Total	1.55
Basin Total	1.86

Element Flows To:

Surface	Interflow	Groundwater

Analysis Results
POC 1





+ Predeveloped x Mitigated

Predeveloped Landuse Totals for POC #1
Total Pervious Area: 1.86
Total Impervious Area: 0

Mitigated Landuse Totals for POC #1
Total Pervious Area: 0.31
Total Impervious Area: 1.55

Flow Frequency Method: Log Pearson Type III 17B

Flow Frequency Return Periods for Predeveloped. POC #1

Return Period	**Flow(cfs)**
2 year	0.005409
5 year	0.018122
10 year	0.036433
25 year	0.080795
50 year	0.139123
100 year	0.23134

Flow Frequency Return Periods for Mitigated. POC #1

Return Period	**Flow(cfs)**
2 year	0.840775
5 year	1.063972
10 year	1.215835
25 year	1.413046
50 year	1.564135
100 year	1.719012

Annual Peaks

Annual Peaks for Predeveloped and Mitigated. POC #1

Year	**Predeveloped**	**Mitigated**
1949	0.004	1.117
1950	0.078	1.141
1951	0.013	0.695
1952	0.004	0.569
1953	0.001	0.640
1954	0.013	0.691
1955	0.002	0.776
1956	0.032	0.741
1957	0.005	0.856
1958	0.006	0.689

1959	0.005	0.718
1960	0.017	0.710
1961	0.008	0.716
1962	0.001	0.622
1963	0.007	0.737
1964	0.020	0.699
1965	0.004	0.896
1966	0.003	0.576
1967	0.049	0.992
1968	0.013	1.177
1969	0.007	0.778
1970	0.001	0.782
1971	0.007	0.920
1972	0.045	0.992
1973	0.001	0.590
1974	0.005	0.872
1975	0.009	0.950
1976	0.014	0.675
1977	0.001	0.701
1978	0.002	0.922
1979	0.001	1.194
1980	0.001	1.128
1981	0.001	0.845
1982	0.009	1.203
1983	0.002	0.975
1984	0.004	0.614
1985	0.001	0.830
1986	0.002	0.728
1987	0.016	1.129
1988	0.001	0.686
1989	0.001	0.985
1990	0.118	1.572
1991	0.040	1.261
1992	0.001	0.605
1993	0.001	0.650
1994	0.001	0.611
1995	0.029	0.754
1996	0.082	0.882
1997	0.013	0.815
1998	0.007	0.796
1999	0.035	1.644
2000	0.001	0.832
2001	0.001	0.915
2002	0.009	1.011
2003	0.003	0.898
2004	0.004	1.550
2005	0.001	0.703
2006	0.013	0.643
2007	0.221	1.460
2008	0.036	1.217
2009	0.002	1.082

Ranked Annual Peaks

Ranked Annual Peaks for Predeveloped and Mitigated. POC #1

Rank	Predeveloped	Mitigated
1	0.2209	1.6439
2	0.1185	1.5716
3	0.0816	1.5501

4	0.0781	1.4604
5	0.0485	1.2614
6	0.0454	1.2166
7	0.0398	1.2026
8	0.0360	1.1943
9	0.0352	1.1772
10	0.0323	1.1408
11	0.0289	1.1285
12	0.0202	1.1282
13	0.0168	1.1166
14	0.0156	1.0825
15	0.0142	1.0114
16	0.0135	0.9924
17	0.0133	0.9920
18	0.0130	0.9848
19	0.0129	0.9751
20	0.0129	0.9496
21	0.0093	0.9217
22	0.0089	0.9195
23	0.0087	0.9151
24	0.0082	0.8984
25	0.0071	0.8964
26	0.0069	0.8821
27	0.0068	0.8719
28	0.0066	0.8564
29	0.0055	0.8449
30	0.0052	0.8320
31	0.0050	0.8296
32	0.0048	0.8146
33	0.0044	0.7960
34	0.0043	0.7824
35	0.0040	0.7781
36	0.0038	0.7761
37	0.0035	0.7543
38	0.0034	0.7412
39	0.0030	0.7366
40	0.0024	0.7277
41	0.0021	0.7179
42	0.0017	0.7157
43	0.0017	0.7103
44	0.0015	0.7029
45	0.0015	0.7010
46	0.0015	0.6992
47	0.0015	0.6948
48	0.0015	0.6914
49	0.0015	0.6889
50	0.0015	0.6856
51	0.0015	0.6755
52	0.0015	0.6502
53	0.0015	0.6426
54	0.0015	0.6398
55	0.0015	0.6219
56	0.0015	0.6137
57	0.0015	0.6113
58	0.0015	0.6051
59	0.0015	0.5905
60	0.0014	0.5762
61	0.0010	0.5692

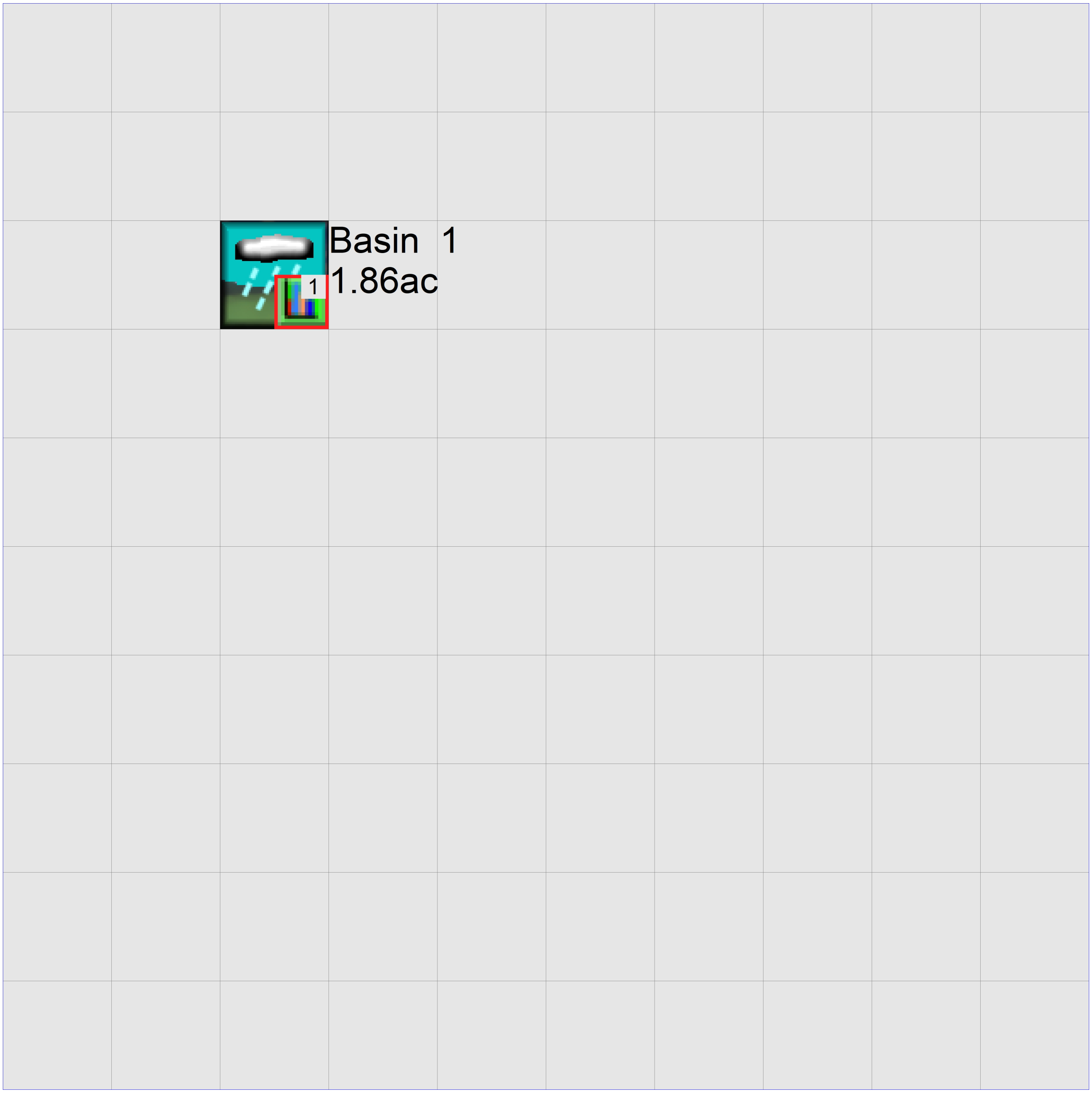
Water Quality

Water Quality BMP Flow and Volume for POC #1

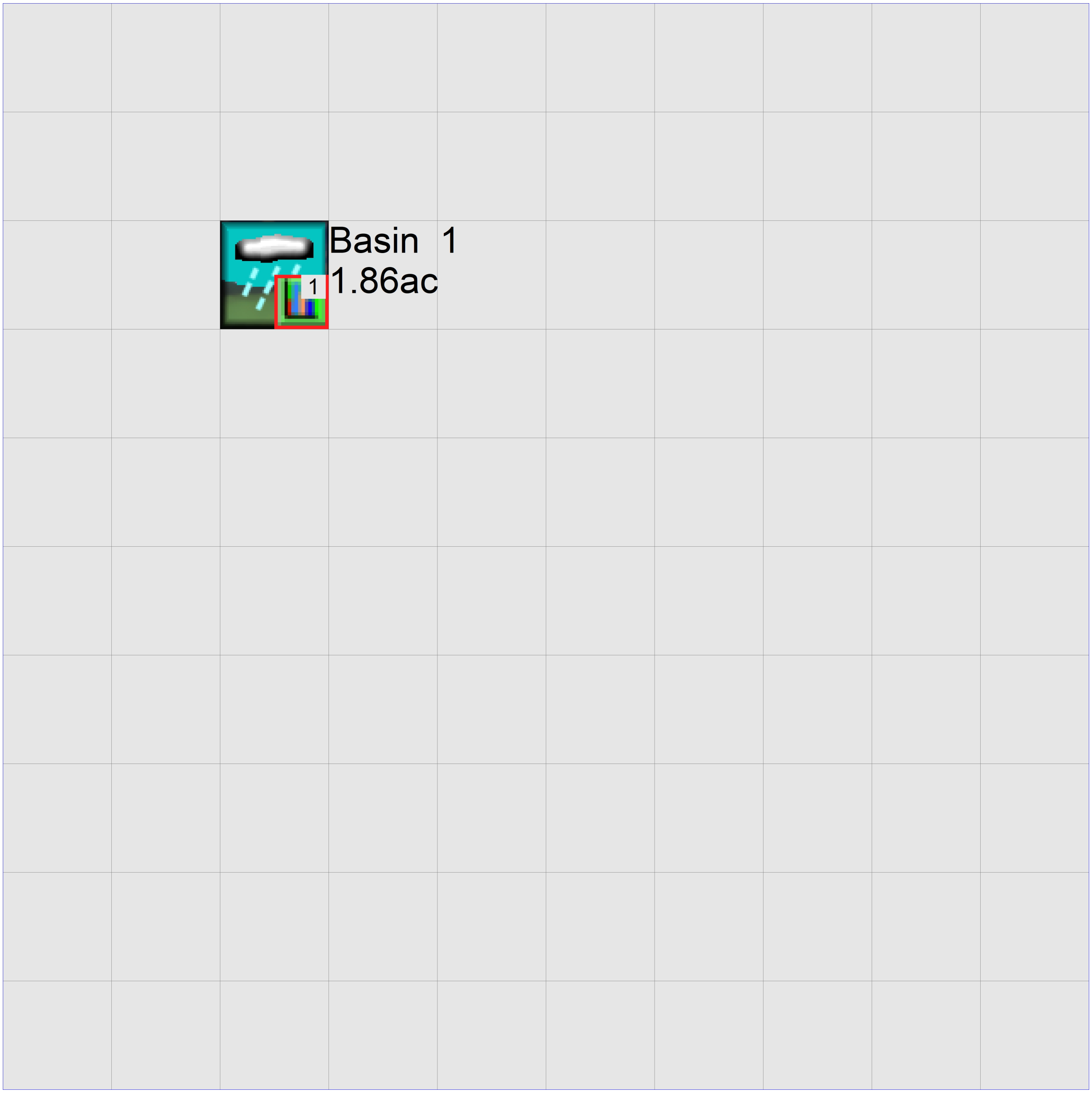
On-line facility volume:	0.2505 acre-feet
On-line facility target flow:	0.3438 cfs.
Adjusted for 15 min:	0.3438 cfs.
Off-line facility target flow:	0.1939 cfs.
Adjusted for 15 min:	0.1939 cfs.

Appendix

Predeveloped Schematic



Mitigated Schematic



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PROJECT REPORT

BASIN 2:
TREATMENT

General Model Information

Project Name:	1883.01 Issaquah Basin 2 Treatment
Site Name:	
Site Address:	
City:	
Report Date:	3/5/2019
Gage:	Seatac
Data Start:	1948/10/01
Data End:	2009/09/30
Timestep:	15 Minute
Precip Scale:	1.333
Version Date:	2018/07/12
Version:	4.2.15

POC Thresholds

Low Flow Threshold for POC1:	50 Percent of the 2 Year
High Flow Threshold for POC1:	50 Year

Landuse Basin Data
Predeveloped Land Use

Basin 1

Bypass:	No	
GroundWater:	No	
Pervious Land Use	acre	
A B, Forest, Flat	0.21	
Pervious Total	0.21	
Impervious Land Use	acre	
Impervious Total	0	
Basin Total	0.21	

Element Flows To:

Surface	Interflow	Groundwater

Mitigated Land Use

Basin 1

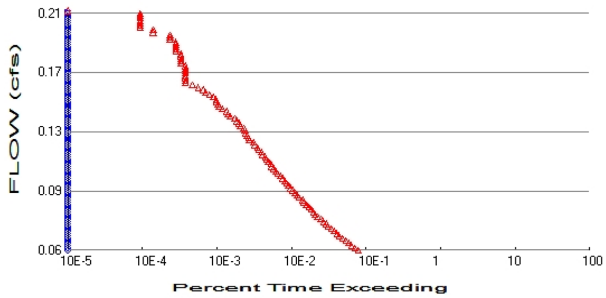
Bypass:	No
GroundWater:	No
Pervious Land Use	acre
Pervious Total	0
Impervious Land Use	acre
PARKING FLAT	0.21
Impervious Total	0.21
Basin Total	0.21

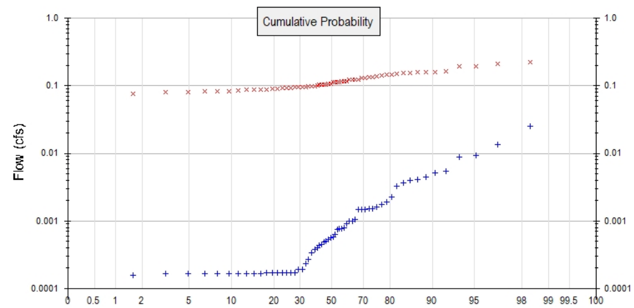
Element Flows To:

Surface	Interflow	Groundwater

Analysis Results

POC 1





\+ Predeveloped x Mitigated

Predeveloped Landuse Totals for POC #1
Total Pervious Area: 0.21
Total Impervious Area: 0

Mitigated Landuse Totals for POC #1
Total Pervious Area: 0
Total Impervious Area: 0.21

Flow Frequency Method: Log Pearson Type III 17B

Flow Frequency Return Periods for Predeveloped. POC #1

Return Period	**Flow(cfs)**
2 year	0.000611
5 year	0.002046
10 year	0.004113
25 year	0.009122
50 year	0.015707
100 year	0.026119

Flow Frequency Return Periods for Mitigated. POC #1

Return Period	**Flow(cfs)**
2 year	0.111706
5 year	0.140855
10 year	0.160633
25 year	0.186262
50 year	0.20586
100 year	0.22592

Annual Peaks

Annual Peaks for Predeveloped and Mitigated. POC #1

Year	**Predeveloped**	**Mitigated**
1949	0.000	0.144
1950	0.009	0.155
1951	0.001	0.088
1952	0.001	0.077
1953	0.000	0.087
1954	0.001	0.090
1955	0.000	0.105
1956	0.004	0.098
1957	0.001	0.112
1958	0.001	0.092

1959	0.001	0.097
1960	0.002	0.093
1961	0.001	0.094
1962	0.000	0.084
1963	0.001	0.096
1964	0.002	0.095
1965	0.000	0.115
1966	0.000	0.077
1967	0.005	0.134
1968	0.001	0.159
1969	0.001	0.105
1970	0.000	0.104
1971	0.001	0.124
1972	0.005	0.124
1973	0.000	0.080
1974	0.001	0.116
1975	0.001	0.129
1976	0.002	0.089
1977	0.000	0.095
1978	0.000	0.125
1979	0.000	0.162
1980	0.000	0.145
1981	0.000	0.114
1982	0.001	0.161
1983	0.000	0.132
1984	0.000	0.082
1985	0.000	0.112
1986	0.000	0.099
1987	0.002	0.153
1988	0.000	0.093
1989	0.000	0.133
1990	0.013	0.193
1991	0.004	0.158
1992	0.000	0.082
1993	0.000	0.088
1994	0.000	0.083
1995	0.003	0.102
1996	0.009	0.113
1997	0.001	0.104
1998	0.001	0.108
1999	0.004	0.222
2000	0.000	0.108
2001	0.000	0.124
2002	0.001	0.136
2003	0.000	0.116
2004	0.000	0.209
2005	0.000	0.091
2006	0.002	0.082
2007	0.025	0.196
2008	0.004	0.152
2009	0.000	0.147

Ranked Annual Peaks

Ranked Annual Peaks for Predeveloped and Mitigated. POC #1

Rank	Predeveloped	Mitigated
1	0.0249	0.2225
2	0.0134	0.2091
3	0.0092	0.1960

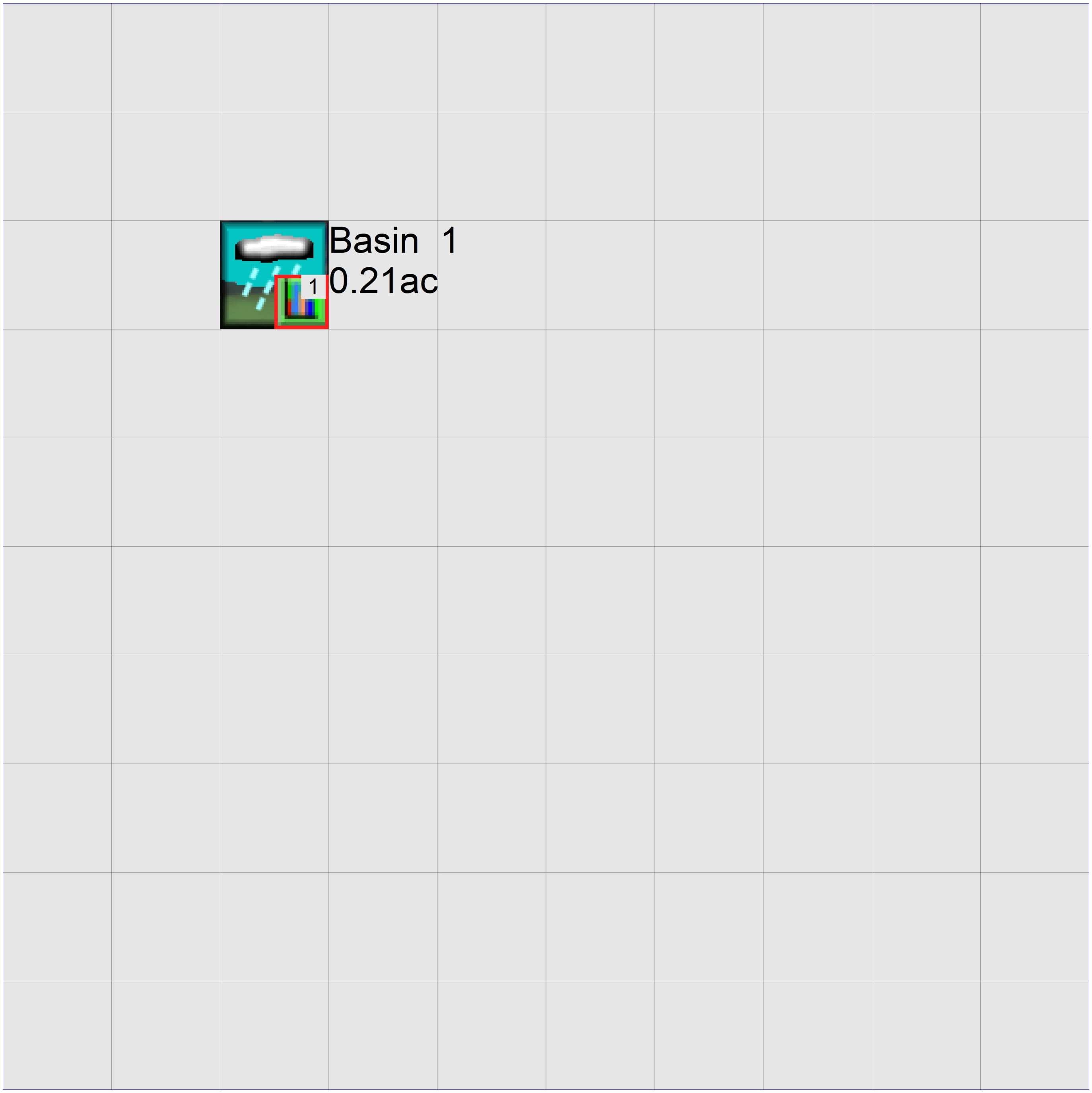
4	0.0088	0.1928
5	0.0055	0.1618
6	0.0051	0.1611
7	0.0045	0.1595
8	0.0041	0.1576
9	0.0040	0.1546
10	0.0036	0.1529
11	0.0033	0.1524
12	0.0023	0.1466
13	0.0019	0.1450
14	0.0018	0.1440
15	0.0016	0.1364
16	0.0015	0.1345
17	0.0015	0.1334
18	0.0015	0.1321
19	0.0015	0.1287
20	0.0015	0.1249
21	0.0010	0.1245
22	0.0010	0.1241
23	0.0010	0.1240
24	0.0009	0.1158
25	0.0008	0.1156
26	0.0008	0.1153
27	0.0008	0.1138
28	0.0007	0.1129
29	0.0006	0.1124
30	0.0006	0.1120
31	0.0006	0.1084
32	0.0005	0.1078
33	0.0005	0.1054
34	0.0005	0.1051
35	0.0005	0.1043
36	0.0004	0.1036
37	0.0004	0.1020
38	0.0004	0.0986
39	0.0003	0.0984
40	0.0003	0.0973
41	0.0002	0.0958
42	0.0002	0.0950
43	0.0002	0.0947
44	0.0002	0.0942
45	0.0002	0.0934
46	0.0002	0.0929
47	0.0002	0.0925
48	0.0002	0.0914
49	0.0002	0.0902
50	0.0002	0.0888
51	0.0002	0.0885
52	0.0002	0.0880
53	0.0002	0.0867
54	0.0002	0.0843
55	0.0002	0.0828
56	0.0002	0.0824
57	0.0002	0.0818
58	0.0002	0.0817
59	0.0002	0.0800
60	0.0002	0.0771
61	0.0001	0.0766

Water Quality

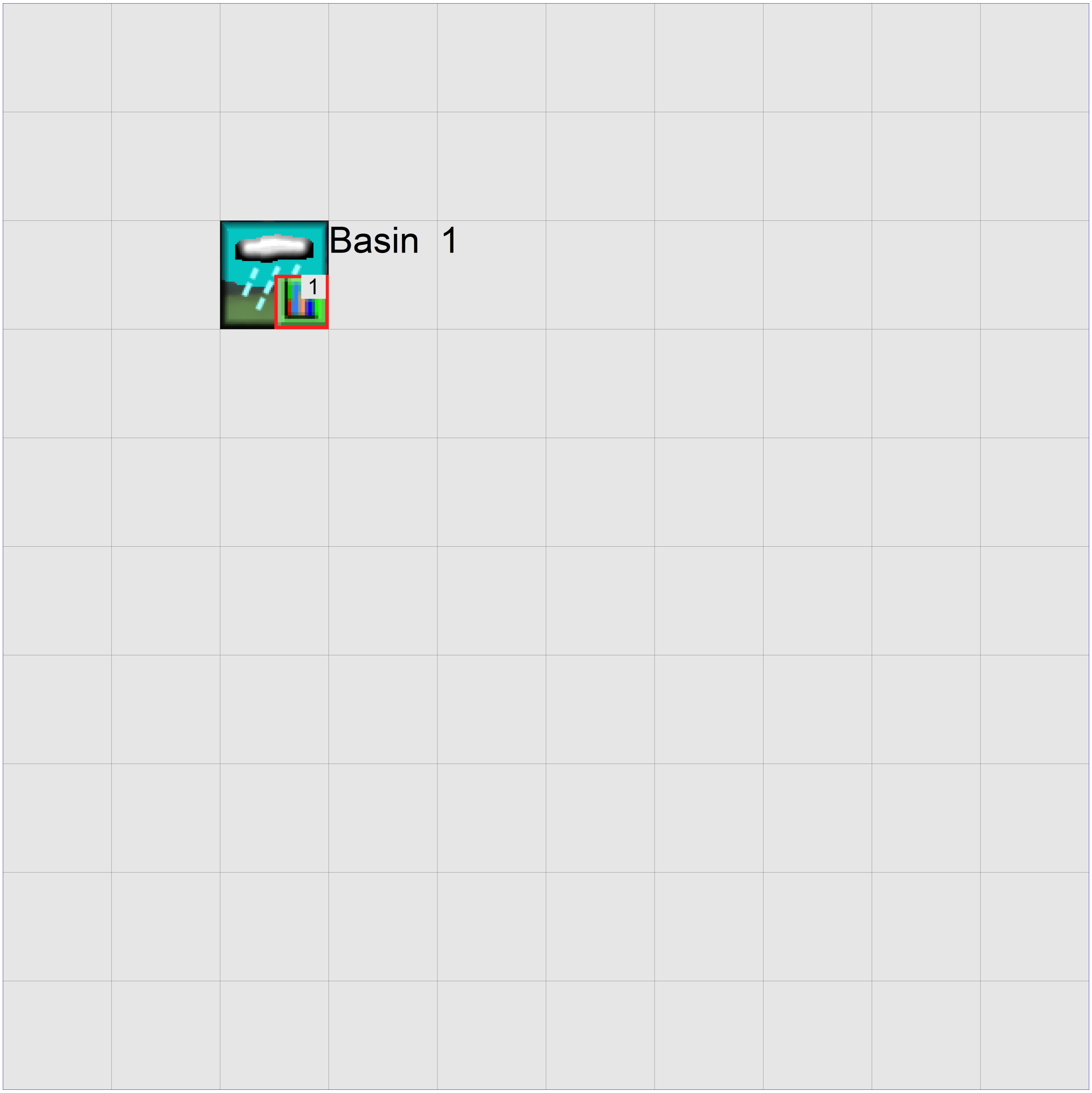
Water Quality BMP Flow and Volume for POC #1

On-line facility volume:	0.0337 acre-feet
On-line facility target flow:	0.0462 cfs.
Adjusted for 15 min:	0.0462 cfs.
Off-line facility target flow:	0.026 cfs.
Adjusted for 15 min:	0.026 cfs.

Appendix
Predeveloped Schematic



Mitigated Schematic



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